JOURNAL

of the

AMERICAN CONCRETE INSTITUTE

Published monthly except July and August by the
American Concrete Institute under the authority of the Board of Direction

EDITED BY THE SECRETARY

with the direction of the PUBLICATIONS COMMITTEE: W. A. SLATER, chairman; P. H. BATES, ARTHUR R. LORD, J. C. PEARSON, A. N. TALBOT, BENJAMIN WILK, HARVEY WHIPPLE, Secretary.

Papers: Original papers, furthering the objects of the society, are invited. They should be addressed to the Secretary, and their consideration by the Publications Committee will be facilitated if: (1) a brief but comprehensive outline or summary precedes or at least accompanies the completed paper; (2) manuscript and photographs are available in duplicate or triplicate; (3) original tracings or other drawings, from which copies are readily made, are available.

Effort will be made to advise authors promptly of decisions as to availability for publication. Acceptance of a paper does not imply the acquiescence of critics (whose advice is sought prior to acceptance) or of the Institute in opinions which it expresses nor the acceptance of data or practice which it records. Institute authority attaches only to standards adopted as provided in the By-Laws. The publication of a committee report implies merely the cooperative effort of an appointed group within the Institute. Acceptance of a paper or report for publication does not commit the Program committee in reference to the annual convention.

Subscription price $12.50 per year payable in advance. (Normally 10 issues—the first volume year will consist of eight issues—November 1929 to June 1930). To members, $10.00 per year, included in the annual dues.

Bound volumes 1 to 22 of *Proceedings of the American Concrete Institute* (1905 to 1926)—$8.00 each so far as available; Volumes 23, 24, 25 (1927-1929) $10.00 each. To members $5.00 each. Volume 26 (1930) consisting of JOURNAL issues 1-8, November 1929-June 1930, bound without *News Letter* pages to non-members and non-subscribers, $12.50. Subscribers and members may obtain the volume (in addition to their monthly issues) at the end of the volume year at $6.00, so long as a limited supply of over-copies permits. Members may insure availability of not more than two copies of each volume at a special price when they place their orders in advance of the first JOURNAL issue of each volume.

Publication address: 2970 West Grand Boulevard, Detroit, Michigan. Entered at the Post Office at Detroit, Michigan, as mail of the second class under the provisions of the Act of March 3, 1879.

Copyrighted, 1930
American Concrete Institute.

AMERICAN CONCRETE INSTITUTE

Founded 1905; incorporated 1906 (National Association of Cement Users); charter amended 1913. (American Concrete Institute.)

BOARD OF DIRECTION

President

D. A. ABRAMS (Term expires Feb. 1931)

Vice-Presidents

S. C. HOLLISTER (Term expires Feb. 1931)
ARTHUR R. LORD (Term expires Feb. 1932)

Secretary and Treasurer

HARVEY WHIPPLE (Term expires Feb. 1931)

Directors:

First District—R. B. YOUNG (Term expires Feb. 1932)
Second District—CLOYD M. CHAPMAN (Term expires Feb. 1932)
Third District—J. C. PEARSON (Term expires Feb. 1931)
Fourth District—P. H. BATES (Term expires Feb. 1931)
Fifth District—F. R. MCMILLAN (Term expires Feb. 1931)
Sixth District—RAYMOND E. DAVIS (Term expires Feb. 1932)

Past Presidents

RICHARD L. HUMPHREY Deceased (1905-14)
LEONARD C. WASON (1915-16)
WILLIAM K. HATT (1917-19)
*HENRY C. TURNER (1920-21)
*WILLIAM P. ANDERSON (1922-23)
*A. E. LINDAU (1924-25)
*M. M. UPSON (1926-27)
*E. D. BOYER (1928-29)

*The five latest, living past presidents are members of the Board.

VOL. 1—NO. 7 MAY 1930 (*Proceedings Vol. 26*)

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

News AMERICAN CONCRETE INSTITUTE Letter

TECHNICAL COMMITTEES

COMMITTEES organized in the last year or for which an Author-Chairman has recently been selected, are listed here so far as these committees and their membership are continued in the present convention year. In a few instances, still under advisement, the complete personnel is not given. Committees are also listed whose tasks are completed and the committee discharged. New committees recently authorized by the Board are specially noted. Other committees will be announced as soon as at least a nucleus of their personnel is determined.

DEPARTMENT 100—RESEARCH

Comprising all committee work in which the principal interest is in the interpretation of test data or the initiation or direction of tests.

Chairman—F. R. McMillan
Co-Chairmen—H. J. Gilkey and F. E. Richart

101—Survey of Current Research

Chairman—H. F. Gonnerman
Secretary—F. E. Richart

J. C. Witt — H. F. Clemmer
F. H. Jackson — F. M. McCullough
D. V. Terrell — J. R. Shank
H. J. Gilkey — C. E. Proudley
F. O. Anderegg

102—Volume Changes in Concrete

Author-Chairman—Raymond E. Davis
Critics—H. J. Gilkey and Charles C. More
(Report published November 1929—Additional work assigned)

103—Effect of Curing Conditions

Author-Chairman—A. A. Anderson

104—Permeability of Concrete

Author-Chairman—L. G. Lenhardt

Charles H. Scholer — M. O. Withey
Ira L. Collier — R. H. Schroeder

105—Reinforced Concrete Column Investigation

Chairman—W. A. Slater

R. L. Bertin — F. E. Richart
R. D. Snodgrass — W. S. Thompson

106—(New Committee) Effect of Vibration on Properties of Concrete

Author-Chairman—W. A. Slater

Present a summary of available information —methods of vibrating; initial and final consistency; proper w/c ratios; use on massive structures, buildings, for high strength columns, in concrete products plants; machines and labor.

DEPARTMENT 200—MATERIALS

Comprising all committee work in which the principal interest is that of the production and distribution of materials used in concrete work.

Chairman—R. W. Crum
Co-Chairmen—E. O. Sweetser and R. W. Johnson

201—Aggregate Specifications

Chairman—H. F. Clemmer

F. H. Jackson — N. C. Rockwood
L. E. Williams — Fred Hubbard
Herb F. Kriege — A. T. Goldbeck
R. L. Bertin — Stanton Walker
H. S. Mattimore — H. J. Love

202—Variations in Standard Portland Cements

Author-Chairman—P. H. Bates

Thaddeus Merriman — T. Thorvaldson
Raymond E. Davis — G. M. Williams

(Report published November 1929—Committee discharged)

203—(New Committee) Cinders as an Aggregate for Concrete

Author-Chairman—Einar Christensen

Present complete summary—proper sources of supply; proper treatment; tests for suitability for use for various purposes, including products and monolithic concrete; proper storage and handling; effect on properties of concrete (as compared with ordinary aggregate) such as weight, strength, E_c; permeability, permanence etc.; effect on reinforcement—in presence of water leaks and in dry locations.

See also November 1929 JOURNAL, *News Letter*, p. 9.

DEPARTMENT 300—ENGINEERING DESIGN

Comprising all committee work in which the principal interest is that of the structural design or of the supervision of construction of such work.

Chairman—A. W. Stephens
Co-Chairmen—Hale Sutherland and Ralph G. Shankland

301—Simplified Rigid Frame Design

Author-Chairman—Hardy Cross

F. E. Richart — Clyde T. Morris
Albert Smith

(Report published November 1929—Committee discharged)

304—Fire Resistance Standards in Design

Author-Chairman—N. D. Mitchell

305—Structural Loads and Reductions in Design

Author-Chairman—R. L. Bertin

George Thompson — Peter Hein
H. N. Howe

306—Permissible Openings in Construction

Author-Chairman—Albert Smith

G. A. Maney — A. E. Lindau

(Progress report published November 1929)

307—Deflections of Reinforced Concrete Members

Author-Chairman—T. D. Mylrea

P. J. Freeman — Fred Kubitz
A. H. Saurbrey

308—(New Committee) Basis of Design for Hurricane Exposures

Author-Chairman—Albert Smith

Discuss: Probable wind velocity and pressures on various types, shapes and sizes of structures; vertical variation in intensity of pressure; suction; sway and gusts of wind; foundation anchorage; stability; method of design, as by lowering stresses or by increasing and properly evaluating loads; effect of neighboring structures; release of pressure by failure of windows, etc., roof signs, etc.

DEPARTMENT 400—ARCHITECTURAL DESIGN

Comprising all committee work in which the principal interest is that of the architectural design or of sculpture or of the supervision of such work.

Chairman—Rexford Newcomb
Co-Chairman—O. F. Johnson

401—Portland Cement Stucco Finishes

Author-Chairman—W. D. M. Allan

Allan Stubbs — C. S. Downs
F. O. Anderegg — Arthur Ware
J. C. Pearson

(Report published November 1929—Committee discharged)

402—Concrete as a Sculptural Medium

Author-Chairman—Lorado Taft

403—Monolithic Concrete Surfaces

Author-Chairman—Nathan C. Johnson

E. K. Borchard — John J. Earley
Nelson L. Doe

(Report received for early publication)

404—Composite Concrete Structures

Author-Chairman—Benjamin Shapiro

405—Economics of Tall Building Design

Author-Chairman—R. R. Zipprodt

S. C. Hollister — A. W. Stephens

(Report presented 26th annual convention—for early publication)

406 (New Committee) Use of Lightweight Concrete in Buildings

Author-Chairman—Frank A. Randall

Discuss: Savings in weight by means of various systems of floor framing; available lightweight concrete—Haydite and cinder concrete—and proper application of each; use of lightweight concrete products for walls and partitions; use as a base for stucco savings in columns and foundations; extra story before changing from spread footings to piles, or from piles to caissons; acoustical and insulating results, etc.

407—(New Committee) Painting on Concrete Surfaces

Author-Chairman—Maximilian Toch

Discuss: Effect of cement, aggregates, oiling of formwork, curing of concrete; effect of condition of surface, rough or smooth, form works, honey comb, sand streaks, porous or dense, etc., effect of drying out vs. painting on green concrete; suction desirable; alkalies, etc. in concrete that would injure or cover up the paint; proper preparation of surface; proper kinds of paint; proper method of application for each kind; proper treatment after applying, for each kind; outdoor and indoor exposures; etc.

DEPARTMENT 500—SPECIFICATIONS

Comprising all committee work in which the principal interest is that of the production of specifications for material, services or workmanship on behalf of the owner.

Chairman—A. B. Cohen
Co-Chairmen—P. J. Freeman and Miles N. Clair

501—Standard Building Code

Chairman—A. W. Stephens
Secretary—R. R. Zipprodt

R. L. Bertin	T. D. Mylrea
H. P. Bigler	F. E. Richart
R. B. Bradbury	Prof. W. A. Slater
Frank E. Brown	W. S. Thomson
M. E. Capouch	L. C. Urquhart
Theo. Crane	N. D. Mitchell
M. E. Hibbs	Hale Sutherland
H. N. Howe	F. R. McMillan
A. E. Lindau	Hardy Cross
Arthur R. Lord	R. B. Young

502—Mixing and Placing Concrete in Buildings

Author-Chairman—Arthur R. Lord

A. S. Douglass	A. B. MacMillan
W. F. Way	J. Thumley

(Report published November 1929)

503—Fabricating and Setting Reinforcing Steel

Author-Chairman—W. F. Zabriskie

W. S. MacKenzie	W. S. Thomson
R. W. Johnson	Arthur R. Lord

(Report published February 1930)

504—Specifications for Ready Mixed Concrete

Author-Chairman—Miles N. Clair

P. J. Freeman	W. E. Hart
R. S. Greenman	C. E. Nichols
Manton E. Hibbs	Alexander Foster, Jr.
Frank I. Ginsberg	H. F. Thomson
	T. C. Powers

(Report published February 1930)

505—Reinforced Concrete Chimneys

Author-Chairman—E. A. Dockstader

506—Concrete Specifications for the Small Job

Author-Chairman—Arthur R. Lord

W. F. Way	P. J. Freeman
Theo. Crane	S. C. Hollister

DEPARTMENT 600—FIELD CONSTRUCTION

Comprising all committee work in which the principal interest is that of monolithic construction, including job equipment, organization, etc.

Chairman—R. B. Young
Co-Chairmen—R. C. Johnson, Nelson L. Doe

601—Measuring Materials for Concrete

Author-Chairman—R. B. Young

John G. Ahlers	A. W. Munsell
A. A. Levison	I. E. Burks
Albert G. Timms	L. G. Lenhardt

602—Design and Operation of Central Mixing Plants

Author-Chairman—Frank I. Ginsberg

A. W. Munsell	N. D. Crowley
H. F. Thomson	Miles N. Clair
G. L. Lucas	Fred C. Wilcox

(Symposium by committee members published March 1930)

603—Developing Latent Strength of Concrete

Author-Chairman—H. F. Gonnerman

604—Winter Concreting Methods

Author-Chairman—R. C. Johnson

W. S. McNaughton	L. J. Street
I. E. Burks	R. B. Young
	Nelson L. Doe

(Preliminary report published February 1930)

DEPARTMENT 700—SHOP MANUFACTURE

Comprising all committee work in which the principal interest is that of the manufacture and distribution of factory made concrete products.

Chairman—W. D. M. Allan
Co-Chairmen—C. F. Buente, C. A. Wiepking

701—Standards for Concrete Masonry Units

Chairman—Benjamin Wilk

702—Prevention of Crazing

Author-Chairman—P. H. Bates

John J. Earley	J. C. Pearson
E. D. Boyer	F. O. Anderegg

703—Color in Concrete Products

Author-Chairman—Raymond Wilson

O. A. Malone	Paul Formigli
	Maximilian Toch

(Report presented 26th annual convention—for early publication)

704—Cast Stone Standards

This committee, with personnel as previously announced, presented its report at the 26th annual convention—published in the *Proceedings* pages this month. For new work the membership is still under advisement.

705—Weather Resistance of Concrete Masonry Units
Author-Chairman—C. A. Wiepking
W. D. M. Allan, Einar Christensen, Dan F. Servey

706—Concrete Pipe

707—Plant Design for Single or Multiple Shift Operation
Author-Chairman—Benjamin Wilk
W. D. M. Allan, D. R. Collins, C. J. Herzog, Henry Parker, A. W. Scheer
(Report published February 1930—Committee discharged)

708—Recommended Practice for the Manufacture of Concrete Masonry Units
Chairman—P. M. Woodworth
M. I. McCarty, C. J. Herzog, W. P. Stanton, Eugene F. Olsen, Stanton Walker

DEPARTMENT 800—USE REQUIREMENTS

Comprising all committee work in which the principal interest arises from the requirement of unusual or special qualities in concrete on account of its use in less common construction or where subject to extraordinary exposures.
Chairman—Raymond E. Davis
Co-Chairman—W. F. Way

801—Durability of Concrete
Chairman—A. E. Lindau
Secretary—F. R. McMillan
R. B. Young, P. J. Freeman, E. Viens, L. W. Walter, G. F. Loughlin

802—Portland Cement Floor Finish
Chairman—John G. Ahlers
Secretary—W. E. Hart
(Report published March 1930—Committee discharged)

803—Disintegration of Concrete
Author-Chairman—G. M. Williams
T. Thorvaldson, John R. Baylis, D. G. Miller
(Report published November 1929—Committee discharged)

DEPARTMENT 900—JOINT EFFORTS

Comprising all committee work done by the Institute jointly with representatives from other societies and work in which the principal interest is clearly in two or more of the foregoing departments.
Chairman—F. H. Jackson
Co-Chairman—Cloyd M. Chapman

901—Concrete Culvert Pipe Standards
Chairman Institute Representation—B. S. Pease

902—Concrete Roads

903—"Joint Committee"—Concrete and Reinforced Concrete
Chairman Institute Representation—S. C. Hollister

CONCRETE PRISMS DOUBLE THEIR 28-DAY STRENGTH

W. SCHNARR*

RECENT changes and extensions to the Queenston Powerhouse of the Hydro Electric Power Commission of Ontario, near Niagara Falls, Ontario, have afforded an opportunity to test concrete of some age.

This powerhouse was built during 1920-1926 and was one of the first large jobs on which the water-cement ratio method of controlling the quality of the concrete was used. The work was described in a paper by R. B. Young presented in 1926 before the American Concrete Institute, entitled "Seven Years of Experience with Job Control of the Quality of Concrete."

The test results of these prisms and cylinders are as follows:

COMPRESSION TESTS OF 6 X 6 X 12 IN. PRISMS CUT FROM CONCRETE AT QUEENSTON POWERHOUSE

Location	Compressive Strength, Lbs. Per Sq. In.		
	Designed	Average Field Test, 28-Day	Field Prisms At Age Of:
Draft tube	1500	1790	3120 / 3470 } 5½ years
Draft tube	2500	2530	6470 / 5870 } 5½ years
Forebay	1500	1740	2800 / 3270 } 7½ years

*Senior Inspector, Hydro Electric Power Commission of Ontario.

The records show that the concrete from which the prisms were cut was placed in the summer, and under ideal conditions.

The 2500-lb. concrete contained approximately 613 lbs. (7 bags) and the 1500-lb. concrete approximately 394 lbs. (4½ bags) of cement per cu. yd. of concrete. Also it might be noted that the 28-day compressive strengths reported for the different groups are the averages of 63, 26, and 276 test specimens respectively.

The draft tube specimens were taken from concrete which was inundated practically as soon as it was placed, and remained under water until the test specimens were taken. The forebay wall cores came from a small retaining wall on the north side of the canal above the water line, and both front and back of this wall were exposed to all weather conditions.

From the above data we note that the cores have more than doubled their 28-day compressive strength.

IMPRESSIONS OF AN INSTITUTE DELEGATE IN JAPAN

BY MAXWELL M. UPSON

THE FIRST World Engineering Congress, held at Tokyo from October 28 to November 7, 1929, established a new standard in lavish and considerate entertainment of convening engineers. Dr. W. K. Hatt and the writer were fortunate in being designated to represent the American Concrete Institute.

Three hundred and sixty-seven engineers, representing almost every country in the world, constituted the official assemblage. With them came wives, sons, daughters and friends, aggregating almost 500 visitors. Of these, the United States contributed the largest delegation—133; followed by Great Britain, 53, and China, 47. Twenty-six countries were represented.

The papers, of which there were almost eight hundred, were classified under twelve divisions, all running concurrently. One of the Parliament buildings was dedicated exclusively to the purposes of the convention, thereby housing all of the academic activities under one roof. There were ten paper-presenting sessions during a period of seven days. Since many of the delegates presenting papers were engineers of prominence, distinguished in their respective specialties, much important engineering information was forthcoming.

English was the official language, although a few papers were presented in French and German by authors who did not speak English.

In addition to those from over-seas, many members were registered from the Japanese engineering fraternity, and of their zealous devotion to the success of the convention, too much can not be said. I attended no session where a spare seat was available after the session was well under way, and in many cases I counted more than fifty standees. This well illustrates the enthusiasm and interest of the Japanese engineer in his profession. He is an indefatigable worker and devoted to the pursuit of knowledge.

Unquestionably the measure of achievement of the convention will be applied more to the social and personal relationship to our Oriental brothers, and to their marked attainments in art and industry, than to the proceedings of the convention. No delegate will forget the courteous consideration and lavish entertainment, the generous facilities and thoughtful planning that made possible the enjoyment of Japan's

rare beauties and outstanding industrial attainments.

Picture after picture lingers in the eye of memory, portraying verdant mountains with their snowy crowns; rushing streams and mirrored lakes; endless fields in spotless order, each square foot cultivated to produce its maximum of life-preserving national food—rice; and the wonders of the inland sea—the glorified Thousand Islands of the Orient. All spell beauty, grandeur and order; happiness and industry; the victory of heroic effort and skillful generalship over an almost cruel dearth of natural wealth. With a population of 400 to the square mile and a tillable acreage of only 15 per cent, it laughs at famine and want that is a gaunt spectre in China with its population of only 175 to the square mile, and a tillable area of 25 per cent.

The miracle is performed largely through industrial and transportation development, and it is of these, perhaps, that we visiting engineers had most to learn.

In 1923, the city of Tokyo, with a population of two and a quarter million, was without one square foot of pavement and had a street area of only one-tenth of its total area. Today it is penetrated by magnificent boulevards in every direction and has a street area of one-fourth of its total area.

Yokohama, a city of more than five hundred thousand, utterly destroyed in 1923, is magnificently rebuilt with wide boulevards and beautiful and imposing public buildings.

The railroads, built and operated by the government on a three-foot six-inch gauge, are maintained on a high standard of operation with modern and efficient equipment. They cost 15 billion dollars and earn 7½% on that sum. After paying interest and depreciation approximately one hundred and twenty-five million dollars are available each year for extensions, improvements and reconstruction.

Almost every phase of industrial production is represented. Factories are modern and imitative in many ways of best United States practice. Many of the most efficient and prosperous manufacturing organizations are controlled by companies jointly owned by the Japanese and the Americans, thereby joining the industry, loyalty and skill of the Japanese with the industrial ingenuity and corporate genius of the American.

The writer returned from China on the steamship "Asama Maru" of the N. Y. K. Steamship Co., the latest production of the Mitsubishi Shipbuilding Co. at Nagasaki. This magnificent steamer of 16,500 tons is equal in equipment, taste and operation to any of our transatlantic facilities; and the Diesel equipment, with its attendant modern auxiliaries, gives an efficiency of operation which compares favorably with any steamers navigating the seas at the present time.

It is quite impossible in a single paragraph to portray adequately the hospitality and entertainment which was accorded to all the delegates and their guests. On our arrival we were presented with our official badges, which were initialed "W. E. C." This we soon interpreted to read: "We Eat Continuously."

Each day, intermingled with the engineering sessions and sight-seeing expeditions, were dispersed sumptuous luncheons, magnificent garden parties, elaborate dinners, dances, theatres and private entertainments. Seemingly, the captains of industry of Japan closed their office desks and devoted ten days

to the entertainment of their visiting guests.

In addition to these hospitable attentions, each member was presented with a two months pass on all the railroads, for himself and his family, together with a very carefully planned schedule embracing thirty-two trips to various points of beauty and interest in the many islands of the Empire. In some cases we traveled in special trains and special cars; again, we traveled singly—all to meet our own personal convenience. It is needless to say that with such facilities, the beauties and charm of Japan were made available to us to a degree that is quite beyond adequate description.

Of the papers presented pertaining to the concrete industry, the following are, in the writer's opinion, of particular interest to the Institute. (Copies of these papers may be obtained by addressing Koseikai Publishing Office, Yuraku-Kan Bldg., Marunouchi, Tokyo, Japan, and enclosing fifty cents to cover cost of paper and postage):

"Cement"—Thaddeus Merriman.
"Some Recent Researches in Fundamental Properties of Concrete in the United States of America"—William K. Hatt.
"On Temperature Changes in the Interior of a Concrete Dam"—Yoshio Kuratsuka.
"Rapid Transit Subways of New York City"—Robert Ridgway.
"Safety in Dam Construction"—Allen Hazen.
"Recent Developments in Portland Cement Manufacture"—Henry Pooley.
"Note on Concreting a Reinforced Concrete Beam and Theory on Corrosion in Steel Reinforced Bars"—S. D. Carothers.
"Cell Concrete"—Erik V. Meyer.
"Works in Masonry and Reinforced Concrete"—Aldo de Dominicis.
"The Effect on the Compressive Strength of Concrete of the Order of Charging the Materials into the Mixer"—Tokujiro Yoshida.
"The Additional Stress Due to Shrinkage in Reinforced Concrete Constructions"—E. Morsch.
"The Development of Theory and Practice in Concrete and Reinforced Concrete"—E. Probst.
"An Investigation of Abrasion in Cements and Mortars"—Shichiro Uchida.
"On the Hardness of Cement with or without Admixtures"—Shichiro Uchida.
"Experiments in Observing Longitudinal and Lateral Strains of Concrete Under Statical and Repeated Compressive Stresses"—Hirohiko Yoshida.
"Recent Developments of Japanese Portland Cement Industry"—Mitsuzo Fujii.
"Experiments on the Permeability to Heat of Reinforced Concrete Chimneys"—Otto Graf.

CHANGES PROPOSED IN CEMENT STANDARDS

RECOGNIZING the fact that a number of large consumers of cement were demanding that it should pass higher requirements than those demanded in the present A. S. T. M. specification, and further realizing that a big majority of the cements being produced materially exceeded these requirements, Committee C-1 having charge of cement standards for the A. S. T. M., has submitted to the Society for consideration at its annual meeting, in June, certain revisions in the present standards.

The tensile strength required of the 1:3 standard sand briquette has been raised from 225 pounds at 7 days to 275 pounds, and from 325 pounds at 28 days to 350 pounds. Further realizing that the methods of tests which form a part of the specification are not as definite as they should be in regard to dimensional limits on much of the equipment used, the Committee is inserting new limits or tolerances applying to most of the equipment used.

The Committee is hoping that the Society will take such action at the June meeting that the standards will be immediately submitted to letter ballot of the Society, so that they may be included in this year's issue of both standards.

To meet the demand for a high early strength portland cement, the Committee prepared a standard covering this cement, differing from the standard for ordinary portland cement only in that at 24 hours the standard 1:3

mortar briquette shall develop a strength of 275 pounds and a strength of 375 pounds at 72 hours. Further, the SO_3 maximum limit has been raised from 2 to 2.5 per cent. This standard has been submitted to the Executive Committee of the Society which in turn referred it to its new Committee 10 on Standards. This Committee has unanimously recommended that this new standard for high early strength portland cement be made a tentative standard of the Society.

IN CONSTANT REPAIR

(Conversation with Dr. Sam'l Johnson)
Dr. Johnson: A man, Sir, should keep his Friendships in a state of Constant Repair.
The Author: And no less, Sir, his Information.

5. ON VOCABULARY

BY ARTHUR R. LORD

MY WIFE is, upon rare occasions, a source of embarrassment to me. As we sit reading by the fire, she may disturb my concentration, and more important, my self esteem, by asking the meaning of some unfamiliar word or foreign expression. While I, naturally, *understand* the purport, I frequently find it difficult to *express* that understanding in anything but the most vague and evasive generalities. It is not so easy to put off one's self on such matters. I have been much annoyed by writers for the Institute's publications, who use terms to me strange and humiliating. I *hope* I have annoyed others in my papers. But that, after all, is a vain hope. We write papers with the intention of putting our experiences and our thoughts before others, to help them, and to help us by their reaction. If we are but imperfectly understood or understood only after unnecessary research, much of our labor may be wasted.

A case in point came up in the November JOURNAL in a discussion by Merriman of a report by Bates. In this discussion occurs an expression "molecular ratio" which I take to be a chemical term, invented since I studied that subject. "A cement with a molecular ratio of 2.60 is an excellent cement but it can be made still better by holding the molecular ratio down to say 2.45." An excellent idea! What the expression "molecular ratio" may mean is of small concern, for so important a quality of cement will doubtless be printed in plain figures on each sack, like the percentage of available nitrogen, potash, etc., on the fertilizer bag. But I did not find it there.

So I wrote to the Editor for a definition, duly cautioning him not to reveal my ignorance. It seems that cement sacks are no better marked in Detroit than in Chicago! But the Editor put me to shame. He immediately wrote to Joe Pearson, asking for the required definition, and adding, "I am unwilling to reveal my ignorance to Mr. Merriman, but you know how little I know!" So I am expecting enlightenment soon.*

This is not so exceptional a case as I could wish. My favorite author among Institute contributors is not Dr. Bogue! If he could *forget* that he is a chemist and *remember* that I am not a chemist! When I seek in his writing to repair my information on cement, I come away with an understanding of why the paste on my envelope flap sticks better if I press on it firmly!

I could demonstrate that my chemical limitations are not my only difficulty in this tender point of un-

*Mr. Pearson's reply immediately follows this article.—EDITOR

explained big words. Some of the admixture "ad" writers are given to embarrassing me. When someone is attempting to *sell* me something, I do not relish being driven to the dictionary in order to comprehend their message. I presume that they are seeking some magic word like "halitosis" to make their sales rival those of Listerine.

A similar, but even more provocative, abuse of my patience has been known to occur in the Institute's *Proceedings*. If a word is in the dictionary (and some technical words are not in mine) I can look it up and only my sense of repose is disturbed. But if some writer refers to Admixtures A, B, C, and D and tells exactly what effect these have on concrete, I am frantic. There is nothing in which I am more interested than in this, and no dictionary is of any avail to help me. If I were chairman of the Publications Committee (Bethlehem papers please copy), I would reject summarily all such perversions of an Institute paper. If Admixture C is beneficial while Admixture D is shamelessly injurious, I want to know, *not* that I am sure to lose if I guess wrong, but how to purchase the good material. What defense is there for such writing? Let this be the end on't!

And I object, too, to using words in a different sense from that in which they are commonly used. The Portland Cement Association has given me invaluable help "on the job" but why will it speak of concrete re-mixed *without* added water (and emphasize its advantages) as retempered concrete (a paper so entitled won the Wason Research Medal of the Institute in 1930) when the man on the job knows that retempering (a common abuse) consists of adding water to mortar, or concrete, that has set until it is too stiff to handle, and *then* re-mixing? I have had such unfortunate pamphlets used against me repeatedly by contractors, on the principle that "you may be able to lick me, but my Big Brother can lick you!" Which is true on most points but not on this one.

But I have sufficiently expanded my theme. It would seem that all I may receive from this article is the sympathy of fellow sufferers, rather than any help from fellow members in keeping our information in a state of constant repair.

ABOUT THE MOLECULAR RATIO*

The "molecular ratio" referred to is simply the old "hydraulic modulus" in different form. The usual way is to give the value of the lime divided by the sum of the silica, iron oxide and alumina, when these are expressed in percentage of total cement—that is, just as they occur in the ordinary analysis of portland cement. Thus in Table II of Committee C-1's 1928 report, (A. S. T. M. Proc. 1928) the values for the 32 cements are given in the line starting "CaO / (SiO_2 + R_2O_3)." In computing the molecular ratio, the same percentage figures for lime, silica, etc., are used, but each percentage is first divided by the "molecular weight" of each oxide, to give, no doubt, a more direct indication of combining of reacting values. The formula which we use is derived from this latter computation, and is given in its simplest form as follows:

$$\text{M. R.} = \frac{\text{CaO}}{.93\,\text{SiO}_2 + .55\,\text{Al}_2\text{O}_3 + 35\,\text{Fe}_2\text{O}_3}$$

Thus if a cement contains 63% CaO, 21% SiO_2, 7% Al_2O_3 and 3% Fe_2O_3, the

$$\text{M. R.} = \frac{63}{.93(21) + .55(7) + 35(3)} = 2.58$$

—J. C. PEARSON

*See foregoing by Mr. Lord on "Vocabulary."

NOTES ON EUROPEAN PROGRAM OF COLUMN TESTS

BY W. A. SLATER*

DR. FRITZ EMPERGER, Chairman of the Austrian Commission on Reinforced Concrete, has written the President of the American Concrete Institute, enclosing an outline for a program of tests of reinforced concrete columns, which it is expected will be carried out simultaneously at Eidgenossene Materialprüfungsanstalt Zurich (Prof. Dr. M. Rós) Technische Hochschule Vienna and Budapest and some other European laboratories, which have similar programs of column tests under consideration.

In view of the fact that the American Concrete Institute is now sponsoring an extensive series of tests of columns, this news is of interest in indicating that there is a live interest in further tests of columns not only in this country, but in Europe as well. The program in Vienna, as stated by Dr. Emperger, is founded on a number of preliminary tests to get at a reliable result. In fact, Dr. Emperger would like also to see tests carried out in an American laboratory along similar lines, for the purpose of mutual comparison.

He points out that in Europe the longitudinal reinforcement in columns is limited to 3 per cent of the sectional area of the columns, and suggests that larger percentages should be permitted with rolled sections. He states that he considers reinforcement with rolled sections "in many respects superior to reinforcement with round iron, for the purpose of compressed sections."

The program includes tests of 50 columns, all of which are 1.5 meters (59 in.) long. Three are structural steel columns, six are cylindrical concrete columns of 23 cm. (9 in.) diameter, made for the purpose of securing direct measurements of the elastic changes of the reinforcing bars, and all the rest are concrete columns 24 cm. (9.5 in.) square. There are four types of columns as follows:

(a) with no reinforcement

(b) with reinforcement consisting of round bars up to 6.4 per cent

(c) with reinforcement consisting of structural shapes up to 9 per cent

(d) steel columns of the same cross-sectional area as the structural shape used as reinforcement for some of the concrete columns.

The columns with no reinforcement are tested to determine the properties of the concrete. A number of tests are made to show whether the shell outside the core adds any strength to the column in case of a low percentage of reinforcement in the form of ties. For this purpose a number of tests are made without longitudinal reinforcement. The columns which have longitudinal reinforcement have also lateral reinforcement, which amounts to approximately 0.5 per cent. In some cases it is in the form of ties, square in plan, and spaced 9 cm. (3.5 in.) apart at the center, and with a closer spacing near the ends of the columns. In others the lateral reinforcement is of circular hooping with a spacing of 2.5 cm. (1 in.) In all cases the lateral reinforcement amounts to about 0.5 per cent. Columns with identical longitudinal reinforcement but with lateral reinforcement of 1 per cent and 2 per cent, respectively, are included. Thus a comparison of the effectiveness of the

*Director Fritz Engineering Laboratory, Lehigh University; Chairman A. C. I. Committee 105, Reinforced Concrete Column Investigation.

different dispositions of the lateral reinforcement will be available.

The longitudinal reinforcement is of two grades, one having a yield point of 2400 and the other 3400 kg. per sq. cm. (34,000 and 48,400 lb. per sq. in.), respectively. Also two concretes are used, the compressive strengths of which are 80 and 240 kg. per sq. cm. (1140 and 3400 lb. per sq. in.).

Evidently Dr. Emperger has confidence in the possibility of securing fundamental data from columns which are much smaller than the majority of columns doing service in engineering structures. This is of much interest to the Institute's committee on column tests, since a large number of their columns are close to the same size as those shown by Dr. Emperger.

In the absence of a statement as to the method of loading, it is assumed that the columns will be tested to failure without holding loads to observe any effect of long continued load. It is evident that this investigation takes quite a different direction from that which is being supported by the Institute. Information along the lines followed by Dr. Emperger's investigation will, however, be very welcome.

(Dr. Emperger, in approving the above, wrote Professor Slater as follows:)

Vienna, April 1, 1930

In possession of yours of March 7, I enclose with many thanks the note you have sent to me. I have made some small changes after having written to Prof. Ros, at Zürich, who has given his consent to this publication. I hope that your support will be useful to the collaboration of all scientific bodies of the world. In every country the ways for such an introduction of tests and the following prescriptions are widely different, and different also are the difficulties for such collaboration. I consider the possibility of a collaboration regarding the column test more from the standpoint of an example for the future. From Germany and France I have received promises. Hungary, for instance, had dispatched to do something of the kind, but independently. Only Switzerland has decided to a strict collaboration in such a way as is not absolutely necessary. The only thing which is important is that the tests remain comparable.

I hope the International Association for Testing Materials and the one for bridges will help to arrive at the necessary understanding. I send you, with the same mail, our report No. 11 on the same subject. Since that we have made some preliminary tests which have been partially published in the technical newspaper. Based on these tests I have made the program of tests which I have sent to you, and I send you now a definite list of the column tests, which will be concreted this day. I enclose also a report explaining which questions will be solved by these experiments. I would like to know for what purpose your 573 column tests are made.

In writing to you it was my desire to get more complete information on the tests made with composite columns in U. S. A. You cannot imagine how difficult it is to get this information in Vienna. There is perhaps in London some library which has a full set of your publications, but since *Engineering News Record* is conducted on the new line and does not contain scientific matters, we have no possibility of getting this information, and I think it will be something the same if you wish to be informed on our work.

You may remember that I came in

1890 to see the World Fair at Chicago and that I stayed in U. S. A. until 1896. I read a paper on concrete-steel bridges in 1894 to the American Society of Civil Engineers and built at the same time the first concrete steel bridges in the United States at the time when no American portland cement was considered to be reliable enough for that purpose. After my return to Europe I tried repeatedly to keep up the scientific connection with the American profession. I published a paper at the World Fair at St. Louis and another one at the Engineering Congress, 1915, in San Francisco. My first tests with columns with rolled sections were started 1902 and published 1908. (Forscherarbeiten aus dem Gebiete des Eisenbetons, Heft VIII, Berlin 1908 "Versuche mit Säulen aus Eisenbeton und mit einbetonierten Eisensäulen.") I have since made about a hundred tests with hooped castiron columns because this was the only "iron" of high-compressive strength which could be had at a reasonable price in the market. In the later years the manufacture of high grade steel had commenced and I dropped consequently the use of cast-iron, and my tests were made with high grade steel. The recent tests are to be made also with high grade steel.

As chairman of the Austrian Commission of Concrete Steel I consider it as one of my principal tasks to improve the collaboration with all other scientific corporations of the same kind. I would be very much pleased to get your help in this direction. Unhappily the congress of concrete steel at Liege (Belgium), 1930, will not be complete so that we have to wait for the congress at Zürich, 1931, (testing material), or at Paris, 1932, (bridges) for an opportunity for a general contact and an oral entertainment on these subjects.

NOTE ERRORS

UNFORTUNATE errors were made in publishing discussions by Prof. T. F. Hickerson and Frank J. McCormick (Report of Committee 301, Simplified Rigid Frame Design) in the April JOURNAL. The last paragraph of Mr. McCormick's discussion should have appeared just over Fig. 2 on page 668—instead it is made to appear as the concluding paragraph of Professor Hickerson's discussion, page 669. Furthermore, the proofreaders turned "statics" into "statistics" at the end of the seventh line, page 669.

NEW MEMBERS

Twenty-five applications for membership in March 1930 have been approved by the Board of Direction from the following:

Blancato, Virgilis, N. Y. Imitation Lime & Granite Stone Co., Inc., 1218 Oak Point Ave., Bronx, N. Y.

Boston Transit Mixers, Inc., 43 Harrison St., Roslindale, Mass. (Myron A. Howe)

Cast Stone & Ornamental Plaster Contractors' Association, 564 Chamber of Commerce Bldg., Los Angeles, Calif. (Donald R. Wadle)

Chapman, John J., 2331 Grandview Ave., Cleveland Heights, Ohio

Glencoe Lime & Cement Co., 901 Syndicate Trust Bldg., St. Louis, Mo. (Guss F. Johannes)

Goss, R. G., 13 Rainsford Road, Toronto 8, Ont., Canada

Gregson, Harry St. J-R-de-Lys, c/o Sir J. Jackson Ld., H. M. Naval Base, Singapore, Straits Settlements

Hansen, Ralph, Central Y. M. C. A., Cleveland, Ohio

Holtz & Co., Inc., R. B., Grand Central Terminal, New York, N. Y. (R. B. Holtz)

Jimenez L., Cesar, 3a, Calle de Granada 82, Mexico, D. F.

Kikkeri, B. E., S. A., c/o T. P. Anantaram, Esq. B. E., S. W. I. Guindy, Madras, S. India

Maryland, University of, Engineering College, College Park, Md. (A. N. Johnson, Dean)

Mitten, Philip J., Mitten Bldg., Philadelphia, Pa.

Noren, Bertil, Dept. Engineering H. C., Cananea, Sonora, Mexico

Patch, Orin G., Bureau of Reclamation, Las Vegas, Nev.

Ricker, Herbert R., River Edge, N. J.

Roberts, F. K., Public Works Dept., Karamea, New Zealand.

St. Louis Duntile Co., Route No. 5, Webster Groves, Mo. (Joseph Amend, Secy.)

Salt River Cement Works, Maitland Rd., Salt River, South Africa

Seron, Levon, c/o Granger & Bollenbacher, 333 No. Michigan Ave., Chicago, Ill.

Smith Chemical Works, Inc., Geo. B., P. O. Box 380, Springfield, Ill. (R. R. McCoy)

Smith, Oscar S., 145 Neal Ave., Dayton, Ohio

Tatnall, Francis G., Southwark Foundry & Machine Co., 400 Washington Ave., Philadelphia, Pa.

Van Name, R. I., Pittsburgh Testing Laboratory, 72 Washington St., New York, N. Y.

Waterside Concrete Works, Commerce & Waterbury Aves., Bronx, N. Y. (Douglas S. Thropp, Gen Mgr.)

STATEMENT OF THE OWNERSHIP, MANAGEMENT, CIRCULATION, ETC. REQUIRED BY THE ACT OF CONGRESS OF AUGUST 24, 1912

Of Journal of the American Concrete Institute published monthly except July and August at Detroit, Mich., for April 1930.

State of Michigan } ss.
County of Wayne }

Before me, a notary public in and for the State and county aforesaid, personally appeared Harvey Whipple, who, having been duly sworn according to law, deposes and says that he is the Editor of the Journal of the American Concrete Institute and that the following is, to the best of his knowledge and belief, a true statement of the ownership, management (and if a daily paper, the circulation), etc., of the aforesaid publication for the date shown in the above caption, required by the Act of August 24, 1912 embodied in section 411, Postal Laws and Regulations, printed on the reverse of this form to wit:

1. That the names and addresses of the publisher, editor, managing editor, and business managers are: Publisher, American Concrete Institute, 2970 W. Grand Blvd., Detroit, Mich.; Editor, Harvey Whipple, 2970 W. Grand Blvd., Detroit, Mich.; Managing Editor, none; Business Manager, none.

2. That the owner is: (If owned by a corporation, its name and address must be stated and also immediately thereunder the names and addresses of stockholders owning or holding one per cent or more of total amount of stock. If not owned by a corporation, the names and addresses of the individual owners must be given. If owned by a firm, company, or other unincorporated concern, its name and address, must be given.) American Concrete Institute, 2970 W. Grand Blvd., Detroit, Mich.

3. That the known bondholders, mortgagees, and other security holders owning or holding 1 per cent or more of total amount of bonds, mortgages, or other securities are: (If there are none, so state.) None.

4. That the two paragraphs next above, giving the names of the owners, stockholders, and security holders, if any, contain not only the list of stockholders and security holders as they appear upon the books of the company but also, in cases where the stockholder or security holder appears upon the books of the company as trustee or in any other fiduciary relation, the name of the person or corporation for whom such trustee is acting, is given; also that the said two paragraphs contain statements embracing affiant's full knowledge and belief as to the circumstances and conditions under which stockholders and security holders who do not appear upon the books of the company as trustees, hold stock and securities in a capacity other than that of a bona fide owner; and this affiant has no reason to believe that any other person, association, or corporation has any interest direct or indirect in the said stock, bonds, or other securities than as so stated by him.

5. That the average number of copies of each issue of this publication sold or distributed, through the mails or otherwise, to paid subscribers during the six months preceding the date shown above is ——. (This information is required from daily publications only.)

HARVEY WHIPPLE

Sworn to and subscribed before me this 24th day of April 1930.

[Seal] ETHEL E. BLAINE

(My commission expires July 12, 1930)

Papers and Reports—In this Issue

Proceedings American Concrete Institute

Vol. 26—Pages 717-798

THE TREATMENT ON MONOLITHIC CONCRETE SURFACES 717
By N. C. JOHNSON, Author-Chairman Committee 403—
Discussion by N. L. DOE

DEVELOPMENTS IN THE MANUFACTURE AND USE OF CONCRETE PIPE 732
By M. W. LOVING

FIRE DAMAGE TO AND REPAIR OF A CONCRETE FACTORY BUILDING 748
By JOHN G. AHLERS

RECOMMENDED PRACTICES IN THE USE OF CAST STONE 760
Report of Committee 704, LOUIS A. FALCO, Chairman

DISCUSSION OF "MOMENT AND SHEAR DIAGRAMS FOR CONTINUOUS BEAMS AND RIGID BUILDING FRAMES" 766

DISCUSSION OF "DESIGN AND COST DATA FOR THE 1928 JOINT STANDARD BUILDING CODE" 768

DISCUSSION OF "DESIGN OF REINFORCED CONCRETE COLUMNS SUBJECT TO FLEXURE" 775

Discussion of the contributions in the following pages is scheduled for publication in the JOURNAL *for November 1930 (except as otherwise noted) and should be available not later than September 1.*

The Treatment of Monolithic Concrete Surfaces

Report of Committee 403, Monolithic Concrete Surfaces

BY N. C. JOHNSON, AUTHOR-CHAIRMAN*

The production of attractive and permanent surfaces on concrete constructions of all kinds has long been the keen desire of those who work in and with concrete.

The subject is of importance, for the more it is studied, the more evident it becomes that the rightful future of cement and cement-concrete depend in appreciable measure upon evolving ways and means to produce surfaces of better initial character and more stability than we have had in the past.

The subject is broad, for there is no branch or class of construction that it does not affect. To deal with it in the confines of a report is not easy, unless a simplification of the subject be made.

As a first simplification, let there be no mental distinction made between concrete in one usage and concrete in another. That is, let us not think of buildings, or any other utility of concrete as being separate, dissimilar or different in character from any other usage insofar as the nature or character of concrete itself is concerned.

If this is done, it will thus be found that industrial floors and dams; bridge piers and ceilings; columns and roadways and other assumedly diverse and different concretes all bear a like relation one to another both in body and in surface; and that like "the Colonel's Lady and Judy O'Grady" they all are "sisters under their skins."

It will at once be observed that this inclusive rearrangement of structures commonly assumed as different brings forward two general classifications of surface. These classifications are: (1)

*Hool and Johnson, Engineers, New York City.

Form-cast surfaces of all kinds and (2) Top surfaces, such as roadways, floors and so on.

For simplicity and clarity, form-cast surfaces will be considered, first and in more detail, both because they present special problems and also because the nature, character and appearance of such surfaces is first in mind with most people when surfacing problems are thought of and discussed.

In form-cast concretes the simplest thing we can hope for is a mere stripping of forms, with a finished article born of the same operation; and as this is one form of concrete surfaces commercially made and sold, let us first examine:

1—FORM-CAST SURFACES

Form-Cast Surfaces, Pointed Up, but Unembellished and Without Covering, present some objectionable features, among which are:

(a) The dull-gray and lifeless color of the form skin.

This skin always is over-rich in cement and lacking in aggregate, both through spading and also because of the natural tendency of cement particles, heavy of themselves, and more agile than other particles because of their size, to go towards and concentrate at the outer, or form surface of the concrete when it is placed.

(b) The constant and resistless and never-ending action and reaction of this surface excess of cement with atmospheric moisture, with corresponding unfortunate volumetric and appearance changes with the progress of the years and the seasons.

This holds true in all but a few localities, much advertised for their balmy climates of almost constant humidity.

(c) The reproduction in this cement form-surface of irregular board lines and of imperfections in forms, such as grain and ends or butting edges of form parts.

(d) Aggravation of the foregoing, caused (in all except steel form work) by the action of the intensely alkaline waters of the concrete combining with the resin of wood, to form expansive resinate of lime, thus expanding first the inner surface of the wood, and causing boards to warp outwardly from the concrete. (Fig. 1.)

This, of course, produces dishings in the concrete which will

be considered further in the next and other divisions; and accentuates the board lines of all kinds in the finished concrete. In later life of forms, this same action produces brittleness of the wood, thus bringing about marrings in the handling and at times, the loss of a form and high costs by reason of short life.

A further aggravation as above is produced in tongue and groove or shiplap form boards by the entry of grout and of alkaline waters into and between the tongue and the groove, thus gradually expanding and soon breaking off, with further

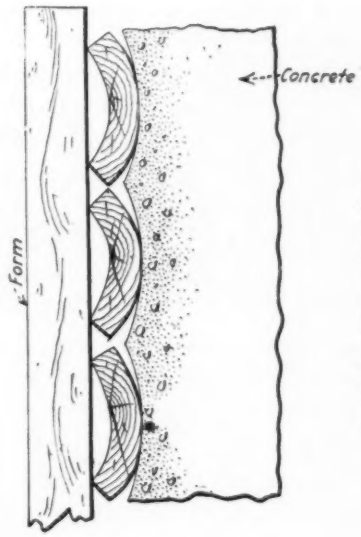

FIG. 1

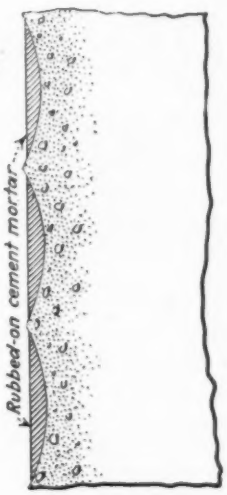

FIG. 2

disfigurement of the concrete, of all such concrete surfaces as have contacted with these forms or parts of forms. (Fig. 2.)

(e) Surface blemishes caused by pouring columns, pilasters, spandrels, curtain walls, etc., in sections and discontinuously, with little puddling, more or less spading and no overflow, leaving distinct pour lines and greater or less accumulations of muck or laitance at these points.

(f) Surface blemishes caused by leaving steel against forms before placing concrete, then prying it backward from the forms

for a distance sufficient only to let "gravy" cover the steel, instead of properly bracing the steel in its designed place.

This gives enough rusting not only to impair the value of the assumedly necessary steel, but also to spall off and to further disfigure by rust stains a surface that might otherwise be good enough for its location.

(g) Disfigurements resulting from spalling, from the removal of tie wires and like minor patchings and from manhandling both concrete and forms in removing forms.

The foregoing remarks are true as of the outward character distinguishing the majority of form-cast concrete. Steel forms bring mitigation of warping conditions as noted above for wood forms, but bring aggravations in others and do not alter the major problem.

In the author's belief, the problem of overcoming the difficulties above listed must be solved, for on its solving depends the rightful place of the concrete art, which, of late years, has been losing ground in percentage place in competition with other building materials.

2—MEANS AND METHODS OF FIXING UP FORM-CAST SURFACES

A careful reading of the foregoing will have made plain a certain emphasis upon the fact that form surfaces in reality are surfaces containing *so large a proportion of cement and so little sand, and no stone,* as to warrant considering them as cement surfaces only.

This thought gives a third simplification of the general problem, for in actuality, it is not surfaces of concrete, but *surfaces of almost pure cement* which we desire to get away from.

3—WASH AND FLOAT WORK

Washing over these form surfaces with grout made of cement, water and greater or less quantities of fine sand and with or without floating, has been done for many years.

Although this treatment has not proven all that could be desired, it is not unexpected that it persists to the present day, for theoretically, "It doesn't cost anything."

Actually, it costs from 4 cts. to 8 cts. per sq. ft. But this would be of no moment as compared to the values conferred if this method gave to concrete acceptable and lasting surface values.

Unfortunately, it does not, nor does it confer clean-cut arrises, acceptable color or any distinction of texture, save and except that it is better, for a time, than the average form-cast surface.

Mitigating against its success are the following factors:

(1) The cement in the wash is as climatically alterable as is the cement in the form skin.

(2) The wash is a thin veneer at best, subject to peculiarly strong internal stresses, as from shrinkages, with changes of temperature and moisture.

FIG. 3—A FORM-CAST SURFACE BEING POINTED UP, BUT UNEMBELLISHED IN ANY OTHER WAY

This illustration is chosen to point out in somewhat exaggerated detail for clearness, the points mentioned in body of report. This is, however, far from a badly done job.

These are tongue and groove boards, but narrower widths, while more expensive, carry the same general characteristics.

(3) The wash lying in the board-formed scallops (See Fig. 2) is a thin crescent, easily loosened and easily removed by the elements.

4—ACCEPTABLE CONCRETE SURFACES AND THE ARGUMENT OF COSTS

Yesterday, today and for many tomorrows the problem of producing acceptable surfaces on concrete will be subject to, hindered by and prevented in large measure by irresponsible statements that have as their big-calibre gun the dreaded and sacred name of "costs."

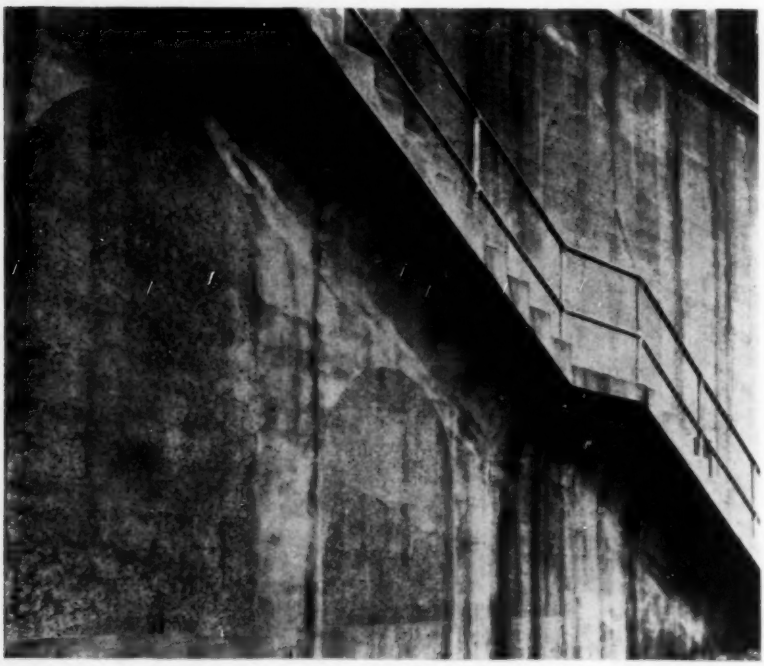

FIG. 4—BUSH-HAMMERED CONCRETE

Illustrating three characteristics—e. g., the lack of values of superficial bush-hammering, even though carefully done; the staining of float and wash work in a short while; and the appearance of irregular pour lines where the concrete is deficient in stone.

There is a price at which work may be well done, a price at which it may be fairly done and a price at which only the sloppiest and most frowzy result can be furnished.

5—Bush hammering or tooling of concrete surfaces

Many attempts have been made on large scale to prepare concrete surfaces mechanically, to encounter the elements and the gaze of the world.

If the concrete is of uniform quality, etc., in an actual rather than an opinionative sense, very good results can be obtained in this way, provided the work is done deeply enough to give texture. (Fig. 7.)

If the concrete is irregular and lacking in actual quality, the result is poor to bad. (Fig. 4.)

And the cost of such work varies from 15 cts. to 60 cts. per sq. ft.

Fig. 5—Deep tooled concrete

An example of the advantages in texture produced by deep tooling, as contrasted with ordinary bush-hammering. Pour lines at arch haunches show through, but the whole surface, made up of light and heavy toolings in contrast, is excellent.

6—Machine grinding of surfaces

Work of this character has been done to a relatively limited degree. Its proponents aver it to be successful. The writer has personal knowledge of only one machine-cut job, and as that concrete was highly irregular, varying from honeycomb to muck,

the result in this instance fell short of being either stone or concrete.

The cost is stated to be around 12 cts. to 60 cts. per sq. ft. for thorough work.

It seems to the writer, that given concrete of good quality, preferably with selected sand that would give color, and with careful placement, the process should lend itself admirably to many desired effects.

7—HAND GRINDING OF SURFACES

Hand grinding with carborundum has not, in the writer's observation, been any sort of success. Where form marks are minor, it is easier. Where form marks are distinct and characteristic as in Fig. 2, the process becomes merely one of a wash-and-float value, with the carborundum hand stone functioning as the float.

8—ACID TREATMENTS OF CONCRETE SURFACES

To remove $\frac{1}{16}$-in. of cement surface by muriatic acid requires, by chemical valence, about $\frac{1}{4}$-in. depth of pure acid.

This makes difficult its successful use in sizable commercial work even aside from its staining effect where metal work is in contact with or adjacent to the concrete.

The cost of using acid effectively is very high. Its use as a wash-over is ineffective as well as costly. As a cleanser, it is good, but can hardly be considered as a true surface treatment or to be recommended for general construction contracts.

9—PAINTING CONCRETE SURFACES

When properly primed with special primers, concrete has been and can be effectively painted.

This subject is too complex to permit of brief treatment. The whole story does not lie in the use of an enamel or other non-saponifying paint. With saponifying oil paints, painting soon deteriorates.

10—VENEERS OF MORTAR OR LIKE SUBSTANCES OVER FORM-CAST CONCRETE

Veneers of appreciable thickness as distinguished from washes, give desirable distinction to form-cast concrete.

Such veneers may be of a wide variety of compositions. Preferably, not less than 2 parts of sand or other aggregate shall be combined with one part of cement. If desired, color may be introduced, or a limited range of tints may be applied later.

Veneers of appreciable thickness give opportunity not only for disguising the blemishes of form-cast concrete, but also give opportunity for the trueing of arrises, the introduction of texture and other characteristics that confer color, texture, vivacity and other desired characteristics to an otherwise drab surface.

FIG. 6—AN ATTEMPT AT BOND

An example of a hacked surface as an attempt at bond, with veneer forced off, in large measure, by efflorescence of the rich form surface left behind the veneer.

The limiting value of veneers is their ability to bond permanently with the form surface of the cast concrete. Experience has shown that although a cement mortar veneer may be applied to the form surface of concrete and although this veneer will stick throughout for a time—perhaps six months in a severe climate and a year or so in a milder climate—loosening and cracking inevitably occur, with possible scaling and loss of value, unless

means are taken to provide on the cast concrete, an adequate bonding surface before applying the veneer.

11—BONDING SURFACES FOR VENEERS OVER FORM-CAST SURFACES

It is recognized that many claim a personal or organization ability to bond anything and everything to the form skin of concrete. But others, on the contrary, disclaim such abilities. For the sake of the many who cannot do this stunt; among whom the writer is numbered, a few ways of providing a bonding surface are set down.

First of all, roughness alone, while an advantage over form-skin surfaces, does not always suffice for permanent bond. The best bond is furnished by roughness plus an exposure of clean aggregate surfaces in profusion.

Bush hammering, so often specified for bond, is, in its usual interpretation, a superficial disfigurement only. It rarely, if ever, provides a good bond.

Cutting by hand tools or machine provides a better bond surface if deeply done, but too often cracks and pulverizes the aggregate in the doing. It also is costly.

Chemical surfacings seem thus far to give the maximum value at the least cost. These will be discussed more extensively in the next chapter.

Acid washing for bond is usually ineffective and is unsuited for the average commercial job for the reasons before given.

12—THE ECONOMIC ADVANTAGES OF VENEERED SURFACES OVER FORM-CAST CONCRETE

It has been proven on more than one hundred and twelve millions of dollars worth of building construction, that it is more advantageous and more economical and permits a maximum of architectural expression and character to:

(1) Provide all parts with adequate bonding surface.

(2) Apply, either as precast units or as run-ornamentation, all cornices, belt courses, pilasters, sills, rustications, etc., in cement mortar compositions, applied directly on the bond-surfaced, form-cast concrete.

(3) Apply the finishing-veneer, true to line, directly to the bond-surfaced concrete.

13—Veneers on form-cast concrete other than cement mortars

Many architects and engineers prefer veneers other than cement-combinations as veneers on concrete.

Space does not permit discussion of them in detail, but their advantages should not be overlooked.

They may be listed in importance as follows:

For exterior use:

Brick alone, or brick with concrete
Terra cotta or terra cotta with concrete
Natural stone over concrete
Cast stone over concrete
Combinations as above and cast iron or aluminum spandrels, etc.

For interior use:

Cement plaster, lime plaster and gypsum plasters

A strange fact is worth noting in this connection, i. e., no engineer and no architect will hesitate to spend money for any of the above materials to hide concrete, but few of them will spend sufficient money—say, in amount, 10% of the cost of any of the foregoing veneers—to supply a real surface in concrete.

The fault, in the writer's mind, for this situation may be laid first, to lack of knowledge in the past; and second to the propagandizing of concrete as a beast of burden alone, with a complete overlooking of its inherent and marvelous abilities that usually lie dormant and unappreciated and that suit concrete to be as and of itself a thoroughly artistic and acceptable surfacing material.

14—Chemical surfacings of monolithic concrete

Chemical surfacings are playing an increasing part in getting away from this situation, yet without increasing the cost.

Their primary function is getting rid of the objectionable and harmful cement form-skin by preventing its set, rather than by the other attempts to achieve the same ends that have been listed above.

Chemical surfacings are applied in two major ways:

FIG. 7—CHEMICAL SURFACINGS

An example of chemical surfacings on formcast concrete using small pebbles as aggregate. Veneer used on swimming pool interior only.

(1) By being painted in a varnish-like coat on forms before concrete is cast.

(2) By being applied directly to concrete, as in top surface work, by means of a hand or power spray. This application is made before the concrete has hardened.

These chemical surfacings are applied either to forms or to top surfaces in a layer that is perhaps 1/50th of an inch thick. This layer does not "eat the cement" in the manner of acids, but reacts with a limited quantity of cement in its formative stage in much the same manner that gypsum does, when added to cement in the making. Without gypsum added at mills, as is well known, all cement would have a flash set. Gypsum holds it back, but does not impair it. Chemical surfacings react still further with a limited quantity of the surface cement.

Whether for bond or appearance the cost of chemical surfaces runs from 2½ cts. to 6 cts. inclusive of all labor and material. The exact cost is dependant on efficiency of operation, rather than on quantity cost.

15—Some examples of chemical surfacing work

Monolithic concrete, with aggregate exposed by stopping the setting of a surface layer of cement as above explained, takes on new character and value when this surface cement is gotten rid of and the better materials of concrete exposed.

Treated in this way, concrete becomes a new material, open to a wide choice of textures, permitting a wide usage of colors and best of all, being no imitation of any other thing.

Fig. 7 as an example of form-cast concrete, shows some uses of this sort of surfaces.

Top surfaces of this kind on roads, pavements and pathways never crack, peel, dust or scale, are cheap to construct, are non-glare and non-skid and any texture, color appearance or combination of these may be obtained.

16—Bond surfaces

Bond surfaces suited to any wage are a wide utility of chemical surfacings. Such bond surfaces never let go of the applied material, whatever its nature.

No apologies are offered for this mention of chemical surfacings. They originated with the writer, it is true, but no report

on surfaces would be complete without mention of them. They deserve recognition and are receiving it commercially in a wider and wider extension of their use.

CONCLUSION

In this report, the endeavor has been to set down, in logical order, what is a known fact only, without praise, blame or partisanship.

Possibly no mention of surfacing concrete and of concrete surfacings will ever, in our generation, be complete without the mention of Mr. John Earley and his specialized and individual work. Quite possibly, no one will ever approach Mr. Earley in his abilities in this field.

None-the-less, the principles followed by Mr. Earley embody at least one of the principles herein set down—e. g., getting rid, always, of the form-skin of cement. And although few, if any of us, are willing to expend enough care and labor to equal Mr. Earley, we could do better than the past average and study this subject earnestly.

BY N. L. DOE*

MR. JOHNSON'S report on the subject of "Treatment of Concrete Surfaces" has sounded a keynote to the industry which must not be ignored if concrete surfaces are to continue to hold their own. Mr. Johnson definitely states in article 13, that various veneers of other materials than concrete are applied to concrete structures at considerable expense on many different types of work, but that an engineer or architect hesitates to spend money to enable a real surface to be obtained in concrete. This situation has been developed to quite an extent recently in the building industry around New York. Much money is now being spent veneering concrete buildings with brick, terra cotta, stone, etc., simply because no concrete surface can be obtained which gives as lasting or as beautiful a surface as other veneers. Unless the concrete industry can develop concrete surfaces which can compete, both in the qualities of beauty and permanence with other materials, the concrete industry cannot hope to find their products specified for finished surfaces by prominent architects and engineers for the better class of work.

*General Superintendent, Turner Construction Co., New York City; critic member Committee 403.

Mr. Johnson has had considerable experience and has developed extensively, certain types of bonding materials and has had more experience regarding the application of cement mortar surfaces to rough concrete than any other members of Committee 403. In article 12, Mr. Johnson has referred to this large experience and presents a statement regarding the economical advantages of veneer surfaces over form cast concrete, which is the result of his own experience rather than the experience of the committee as a whole. From personal experience, it is impossible for the writer to state that the processes as discussed in article 12 can be economically handled on general work or not. The readers of this paper must realize that these statements are almost entirely the result of Mr. Johnson's personal observation.

Readers are referred to the JOURNAL *for November 1930, for discussion which may develop. Such discussion should reach the Secretary by Oct. 1, 1930.*

Developments in the Manufacture and Use of Concrete Pipe

BY M. W. LOVING*

Two general classes of concrete pipe are produced in this country—plain and reinforced. Plain concrete pipe was first manufactured in the United States with natural cement and fine aggregate in 1842. Specimens examined in recent years indicate that the concrete was placed in forms by the hand tamping method, utilizing a dry consistency of concrete which would permit the immediate removal of the forms. The design of the pipe included in most cases a flat base cast integrally with the pipe. In many cities, this old pipe has served in sewers, drains, etc., to the present day. The durability of the pipe has varied, depending on the care and ingenuity employed in its manufacture. The principal cause of failure was the use of an insufficient quantity of mixing water in the concrete which, together with inadequate curing, would not bring about complete hydration of the cement. Because this class of pipe was usually made with cement and fine aggregate, it was customarily referred to as "cement pipe."

Reinforced concrete pipe was first developed in France between 1895 and 1900 and was introduced in this country in 1905. Some of this early pipe was fabricated with a comparatively dry mix of concrete, consisting of 1 part of portland cement, 2 parts of fine and 4 of coarse aggregate. In many cases, bank run aggregates were used, as was true of concrete used in other structures of that day. Later a wetter mix of concrete was employed. The pipe was cast in steel forms which consisted of an inner and outer casing with base rings and headers of cast iron, both of which formed the joint. The concrete used in reinforced concrete pipe was not materially different from that used in other structures.

*Sec'y American Concrete Pipe Association

Consequently, such pipe has given good service for the principal reason that an adequate amount of mixing water was always used to hydrate the cement. Besides, the necessity for curing was not as important as in the case of the dry mixed, tamped, plain pipe.

PLAIN CONCRETE PIPE

Plain concrete pipe today is manufactured by machinery in diameters from 4 to 24 in. and larger. The necessity for quantity production has practically eliminated hand tamping or cast methods for this pipe. The minimum shell thickness for 4-in pipe is $\frac{9}{16}$ in and $2\frac{1}{8}$ in. for 24-in. pipe. Consequently, the concrete must be placed under great pressure in order that the pipe meet rigid specification requirements. Two general classes of machinery are employed, operating on the packerhead and tamping principles.

The packerhead machines, by means of wing packers and a revolving, troweling head, press the concrete outward against an outer form. The outer form, which remains stationary, with its greater frictional area, causes the concrete, as it is fed automatically into the form in thin layers, to remain stationary. The packers work outward and up and the machine must be adjusted so that the concrete is not fed into the form too rapidly. Otherwise, seams or rock pockets will appear in the finished pipe and the concrete may "climb" ahead of the packer with the same results. Concrete of semi-dry consistency is used, allowing removal of forms as soon as the pipe, in its form, is placed in the curing room. If too much mixing water has been used, the pipe will warp or collapse on removal of the form. If not enough water has been used, the distinct web-like markings on the exterior surface of the pipe will not appear.

Machines operating on the tamping principle consist of a stationary core which also trowel the inner surface of the pipe, and an outer casing which revolves as concrete of semi-dry consistency is fed automatically into the form. The greater surface and frictional area of the outer form causes the concrete, which is tamped uniformly in thin layers, to revolve about the inner core. If the concrete is fed too rapidly, seams and rock pockets will appear in the finished pipe. When the pipe is finished, the inner

core is elevated and removed by an automatic device and the new pipe in the outer form is conveyed to the curing room and the form removed at once.

Mix and Gradation of Concrete. Gradation of the mix of concrete is dependent on the size of pipe manufactured. In 6-in. pipe, the coarse aggregate used seldom exceeds a ⅜-in. maximum, while in 24-in. pipe, ¾-in. coarse aggregate is often employed.

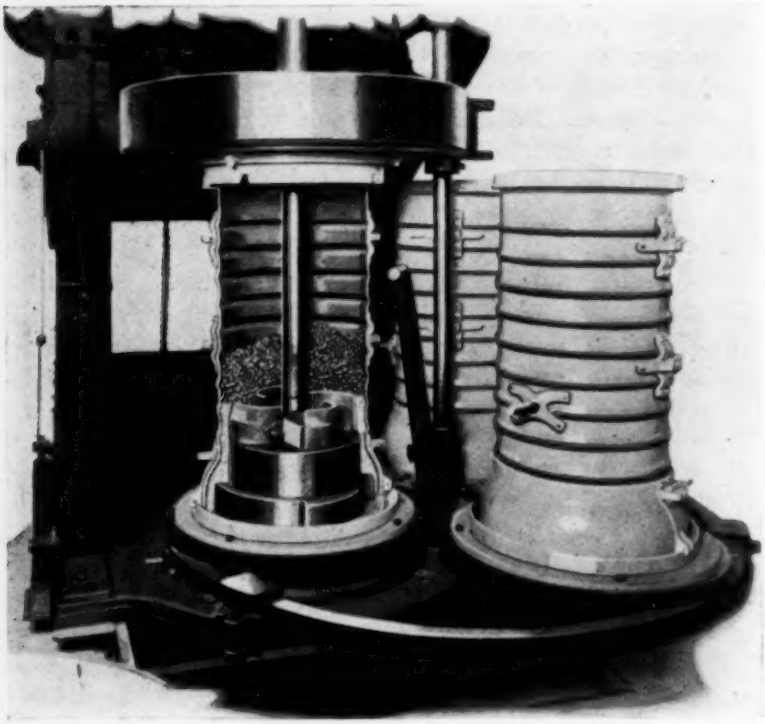

FIG. 1—MOLDS, PACKERHEAD—INCLUDING WING PACKERS, TROWELING HEAD AND BELL PACKER. BOTH PLAIN AND REINFORCED PIPE WITH DIFFERENT TYPES OF JOINTS ARE PRODUCED BY THIS PROCESS

Since plain concrete pipe is made to meet the requirements of the Standard Specifications for Cement-Concrete Sewer Pipe of the American Society for Testing Materials, the manufacturer has to pay strict attention to the quality and gradation of aggre-

gates for each diameter of pipe. Three tests are required—the strength test (external loading), the absorption test (5-hr. boil) and the internal hydrostatic pressure test (percolation). The more coarse aggregate used, the greater the strength and the

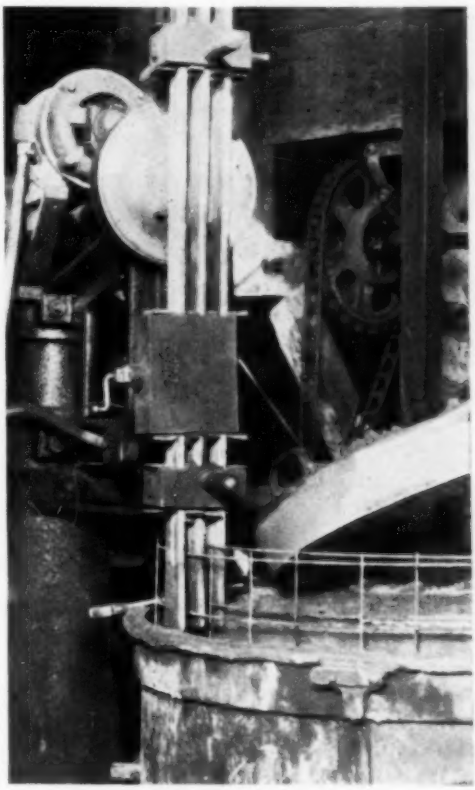

FIG. 2—MULTIPLE TAMPERS EMPLOYED IN THE MANUFACTURE OF DOUBLE LINE REINFORCED CONCRETE PIPE. IN THIS ILLUSTRATION, THE STEEL CAGES WERE RAISED FOR THE PHOTOGRAPH. REINFORCED AND PLAIN CONCRETE PIPE WITH DIFFERENT TYPES OF JOINTS ARE PRODUCED BY THE TAMPING PROCESS.

lower the absorption. Unless the grading of the aggregate is carefully worked out and the concrete mixed for a sufficient length of time, the pipe may fail to meet the percolation test because

water often percolates through crevices among the coarse aggregate.

To meet the three tests, the manufacturer must test the pipe at frequent intervals and experience will enable a manufacturer to produce pipe of unusual strength and density. A minimum absorption of 8 per cent is required. In the percolation test, the pipe must withstand without leakage, 5 lb. per sq. in. for 5 min., 10 lb. for 10 min., and 15 lb. for 15 min. The strength test per foot increases with the diameter of pipe.

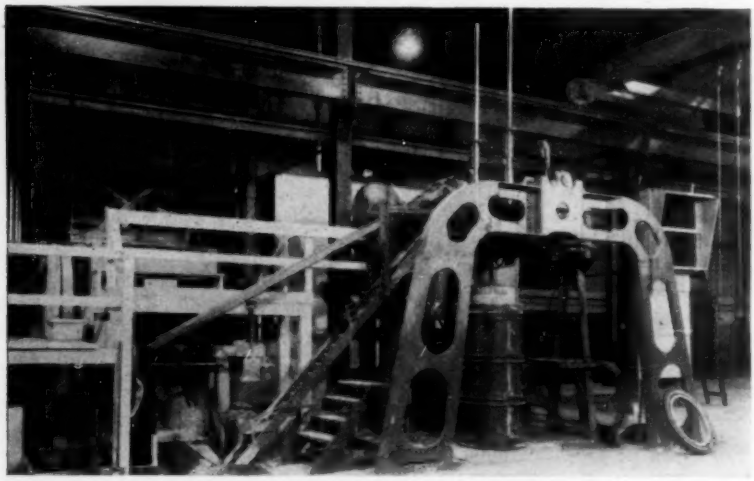

FIG. 3—ANOTHER TYPE OF TAMPING MACHINE FOR THE MANUFACTURE OF REINFORCED CONCRETE PIPE IN DIAMETERS PRINCIPALLY FROM 24 TO 60 INCHES. PIPE WITH BOTH DOUBLE AND SINGLE LINE REINFORCEMENT IS FABRICATED WITH THIS TYPE OF MACHINERY.

It is customary to exceed specification requirements and in many cases, 6-inch plain concrete pipe with a shell thickness of only ¾-in. will withstand from 50 to 100 lb. per square inch internal hydrostatic pressure without percolation of water through the thin shell. This applies to pipe made by either the packerhead or tamping process and shows what results are obtained by the use of only portland cement, fine and coarse aggregate and water.

Curing. Curing is of the utmost importance for pipe made by the tamping or packerhead processes. Either steam or water-spray or a combination of both is employed, depending on climatic and economic conditions. Plants operating in cold climates use steam in winter and water spray in summer. The only advantage of steam curing over water spray is the higher temperature, which accelerates the hardening of the concrete. Curing rooms are arranged so that air currents cannot circulate within them and the time of steam or water spray curing at a given temperature varies from 48 to 72 hours or until the tests show that the pipe will meet the specification requirements after it has been seasoned in the storage yard.

USES

Sewers. Plain concrete pipe is extensively used for the construction of sanitary, combined and storm sewers in many cities of the United States and Canada. Portland cement mortar is generally used for making the joints and experience has shown that with a reasonable amount of care on the part of the pipe layer, the joints can be made watertight to prevent leakage, infiltration and the entrance of tree roots into the finished line. Other types of jointing materials are used where preferred or specified by the engineers.

Drainage. Concrete drain tile, which is required to meet the Standard Specifications for Drain Tile of the American Society for Testing Materials, has been and is extensively used for farm drainage, as well as the under-drainage of highways. No provisions are made for joints in drain tile since they are left open to facilitate infiltration.

Irrigation. Vast quantities of concrete pipe are used for the irrigation of land, principally in California, Washington, Oregon, Colorado, Texas and recently in Florida. There are no national specifications covering pipe for this purpose. It is produced, however, to comply with the A. S. T. M. standard specifications for plain concrete sewer pipe but, generally speaking, more attention is paid to the internal hydrostatic pressure test than the other two tests required for sewer pipe. The joints for irrigation pipe are of the special bevel type, even in the very small sizes. The joints are made with portland cement mortar and bonded

with the same material. The consistency of the mortar is comparatively wet—so the mortar will adhere to the joint when troweled. By careful laying and backfilling, a pipe line so made will withstand at the joints the same pressures as required for the barrel of the pipe.

Culverts. Plain concrete pipe is used for culverts to a limited extent in the smaller sizes 12 in. to 24 in. and is made to meet, or exceed, the A. S. T. M. requirements for plain concrete sewer pipe. The walls are generally thicker and approximately the same as for reinforced concrete pipe, described below.

Electrical Conduit. Plain concrete pipe is produced and used on a very large scale in Chicago and vicinity for the construction of electrical conduits. These pipe are made by the packerhead process in 3-ft. lengths and diameters of approximately 4 in. When the pipe are laid in batteries, a metal band or sleeve is placed over the joints and the pipe lines are spaced so that the proper thickness of monolithic concrete encasement can be made. This pipe is also produced and used in the San Francisco district of California by electrical companies, and elsewhere.

REINFORCED CONCRETE PIPE

Reinforced concrete pipe is manufactured today by three general processes—cast, machine and centrifugal.

Cast Process. Forms for reinforced concrete pipe produced by the cast process consist of steel inner and outer casings, headers and base rings which form the joints. Diameters from 24 to 108 ins. in lengths of 4, 5, 6, 8 and 12 ft. and longer are produced by this method. The concrete used varies from a quaky consistency to comparatively dry mixes. Mixes from 1:2:4 to 1:1½:2 are used, depending on service and test requirements. Concrete of dry consistency is placed either by jolting the form as it is filled or by the use of electric vibrators. Where the wetter consistencies are used, the concrete is carefully spaded in thin layers.

Ordinarily the pipe is made in the presence of an inspector representing the user. Cylinders are made and cured under identical conditions with the pipe and tested. This, together with an accurate check of the amount and placement of steel reinforcement, constitutes the main program of inspection. Pipe are sometimes tested for external loading.

Machine Process. Reinforced concrete pipe is produced by the tamping machines in diameters from 12 in. to 60 in. and by the packer-head process from 12 in. to 30 in.

For the larger pipe, there is another type of tamping machine. A form consisting of an inner and outer casing revolves on a platform and concrete of semi-dry consistency is tamped in thin layers. The finished pipe in the form is removed from the machine to the curing room, where both the inner and outer casings are

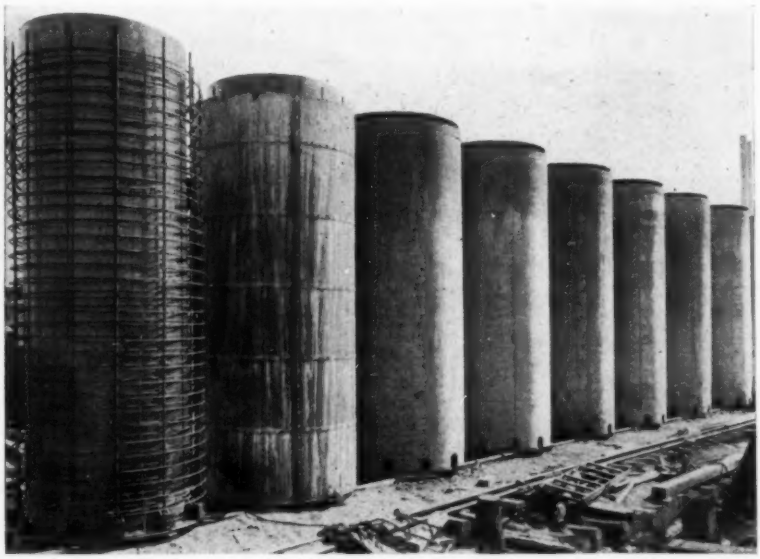

FIG. 4—MANUFACTURING 72-INCH REINFORCED SUBAQUEOUS PIPE BY THE CAST PROCESS FOR THE CLEVELAND, OHIO, OUTFALL IN 1916. THESE PIPE WERE 18 FEET LONG. THE 84-INCH PIPE FOR THIS OUTFALL WAS 20 FEET LONG

removed at once and re-used. In some of the larger pipe, it is necessary to extract the inner core before the pipe is placed in the curing rooms.

Pipe is made by machine in diameters from 12 to 60 in. and larger and in lengths of 3, 4 and 6 ft.

Centrifugal Process. This process has been used for many years, principally in Australia, South Africa, New Zealand, England, Germany and France and was introduced in this

country and Canada since 1920. Concrete is automatically fed into forms consisting of an outer casing which revolves (horizontally) at various peripheral speeds, slow at first and increasing after the form is filled. Lateral movement of the concrete is prevented by the use of end rings which also form the joint. If too wet a mix is used, separation of the coarse aggregate may result, particularly if the particles vary in specific gravity. This will cause roughness of the interior surface of the pipe. The centrifugal

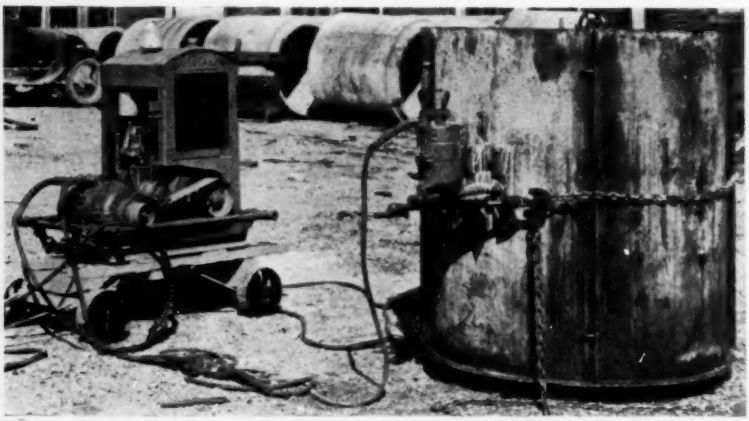

FIG. 5—MANUFACTURING REINFORCED CONCRETE SEWER PIPE BY THE CAST PROCESS, UTILIZING ELECTRIC VIBRATORS. PORTABLE POWER UNIT SHOWN AT LEFT

action expels excess water and light materials in the form of slurry.

Several methods are used for finishing the interior surface of the pipe and in some cases, means are provided for compacting the concrete during the spinning operation.

The forms for centrifugal pipe are of two principal classes—solid and in sections. In the sectional form, the finished pipe is generally steam cured to accelerate the hardening of the concrete so that the form can be re-used. The form is removed by releasing the locks holding the sections.

The solid form in some cases has an internal diameter slightly greater at one end than at the other; others have uniform diam-

eters. The interior surface of the form before use is coated with a thin layer of paraffin, wax or a similar material, which is allowed to harden. The concrete is then placed as described above and the form, with the finished pipe, is steam cured. The paraffin or wax melts, is later reclaimed, and the pipe removed from the form with comparative ease.

Steel Reinforcement. Steel reinforcement consists of wire which meets the requirements of the current specifications for cold-drawn steel wire for concrete reinforcement of the American

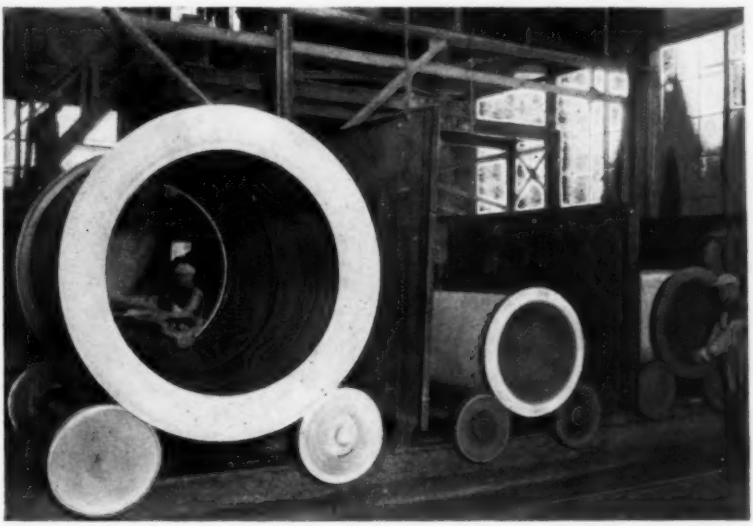

FIG. 6—MANUFACTURING REINFORCED CONCRETE PIPE BY THE CENTRIFUGAL PROCESS, IN WHICH SECTIONAL MOLDS ARE USED

Society for Testing Materials or of bars which meet the requirements of the current standard specifications for billet-steel for concrete reinforcement bars of the American Society for Testing Materials. Each line of circumferential reinforcement is assembled into a cage and must have sufficient longitudinal bars or members extending through the barrel of the pipe to afford rigidity and maintain the reinforcement in exact shape and correct position within the form.

When a single line of circular reinforcement is used in circular pipe, it is placed approximately 1 inch from the inner surface. When two lines of reinforcement are used in circular pipe, one is placed approximately 1 inch from the inner and outer surfaces respectively. When a single line of elliptical reinforcement is used in circular pipe or a single line of circular reinforcement is used in elliptical or oval pipe, it is placed approximately 1 inch from the outer surface at the springing line (horizontal line through the center) and approximately 1 inch from the inner

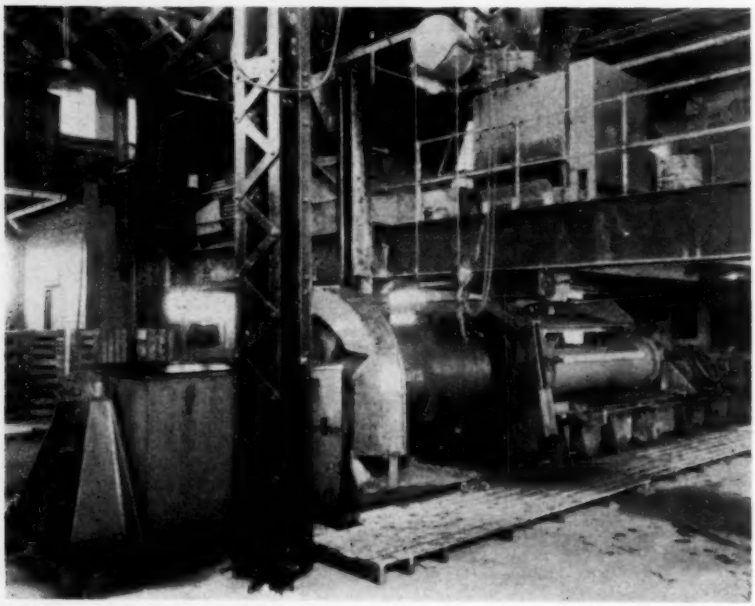

FIG. 7—SOLID MOLDS ARE USED IN THIS CENTRIFUGAL PROCESS FOR MANUFACTURING CONCRETE PIPE. A TROWELING AND COMPACTING DEVICE IS ALSO EMPLOYED

surface at the crown and invert. This description applies to pipe when laid. It is necessary when elliptical reinforcement has been used to designate by markings on the surface of the pipe the correct position when laid.

The circumferential steel members must be lapped at least 30

diameters or if welded, are required to develop the full strength of the wire.

Spiral wound cages are fabricated on special machines for both elliptical and circular reinforcement. The longitudinal members are either securely tied or are spot-welded.

USES

Sewers. Reinforced concrete pipe is extensively used for the construction of sanitary, combined and storm sewers from 24 to 108 in. internal diameter. Pipe is produced by the processes just described to comply with the Tentative Standard Specifica-

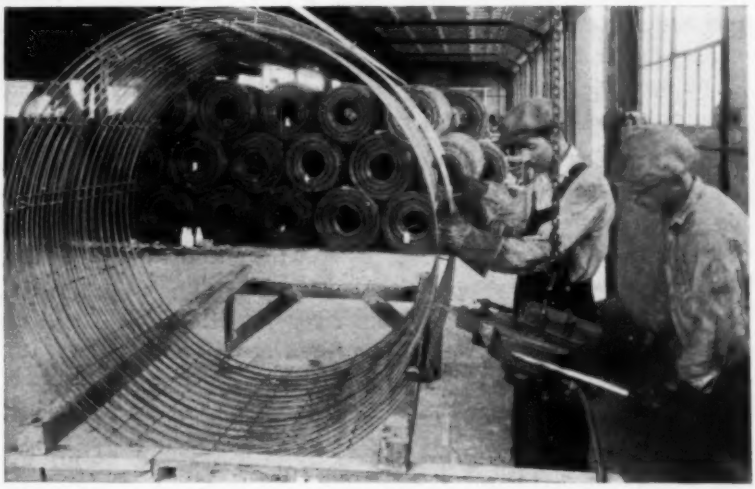

FIG. 8—PRECISE METHODS USED FOR PLACING STEEL REINFORCEMENT IN CONCRETE PIPE

tions for Reinforced Concrete Sewer Pipe of the American Concrete Institute. These specifications cover pipe from 24 to 72 in. inclusive and in the sizes 78 to 108 in., manufacturers' or special specifications are complied with. A number of different types of joints are employed and it is customary, owing to the difficulty of describing these joints, simply to specify the engineering requirements for the joint. The following is quoted as typical from the specifications for reinforced concrete sewer pipe of the American Concrete Institute:

The ends of the reinforced concrete pipe shall be so formed that when the pipe are laid together and the joints cemented, they shall make a continuous and uniform line of pipe with a smooth and regular interior surface. The joints shall be of such a design that when cemented they will prevent leakage and infiltration as well as appreciable irregularities in the flow line of the sewer.

Culverts. Reinforced concrete pipe has been and is used on a large scale by the principal railroads and state and county highway departments of this country for the construction of culverts. It is made in standard sizes from 12 in. to 84 in. and in lengths of 3, 4, 6 and 8 ft.

The research work by Prof. A. N. Talbot of the University of Illinois, described in Bulletin 22 entitled "Tests of Cast-Iron and Reinforced Concrete Culvert Pipe," published in 1908, was sponsored by five of the principal railroads of the middle west. This formed the basis for the development of reinforced concrete pipe for culvert and sewer construction in this country.

In 1919, the Joint Concrete Culvert Pipe Committee was organized. The personnel of the committee consisted of two representatives each from the following organizations: American Society for Testing Materials; Bureau of Public Roads, U. S. Department of Agriculture; American Society of Civil Engineers; American Association of State Highway Officials; American Railway Engineering Association; American Concrete Institute; American Concrete Pipe Association.

At the outset, the committee found that there was little known with reference to the actual loads to which culverts are subjected. An investigation of this subject had been inaugurated at Iowa State College, under the direction of Dean Anson Marston, chairman of the committee, and the committee encouraged the vigorous prosecution of that work so that results could be used in connection with the development of specifications for reinforced concrete culvert pipe. Two progress reports on this investigation were submitted to the committee and the data were of great value in the determination of loads for which culvert pipe should be designed. Besides, a considerable amount of data from tests of pipe and from experience in the use of pipe was furnished by members of the committee and by manufacturers of pipe and engineers who had installed the pipe or observed its behavior in embankments. The first progress report of the com-

mittee was issued in 1926 and it was the general opinion of many of the manufacturers that the steel specified was excessive. Consequently, 167 standard sections of concrete pipe, varying in diameter from 12 to 60 in. were manufactured in the presence of and tested approximately 28 days later by a special representative of the Joint Committee. These tests showed that pipe of a specified strength could be made with less circumferential reinforcement than was required by the specifications in the 1926 report. Two standard classes of pipe are included in the 1928 specifications,* known as standard reinforced concrete culvert pipe and extra strength reinforced concrete culvert pipe. The work of the Committee was conducted on a high engineering plane and the research work included a considerable financial outlay.

Irrigation. The United States Reclamation Service has used reinforced concrete pipe on an extensive scale for the construction of irrigation lines. This likewise applies to many individual land developments in the states mentioned where plain concrete pipe has been used for this purpose.

It is customary with reinforced concrete pipe to figure the working stress of steel reinforcement at approximately 12,000 lb. per sq. in., the tensile strength of the concrete being considered as an additional factor of safety. A number of irrigation lines have been constructed where internal hydrostatic pressures up to 100 lb. per sq. in. are provided. This subject is discussed in detail in Bulletin 852 entitled "The Flow of Water in Concrete Pipe," published by the U. S. Bureau of Public Roads in 1920.

Water Supply. In the construction of modern and efficient aqueducts, generally of the diameters above 24 in., reinforced concrete pipe is today extensively used. The steel reinforcement is generally designed on the same basis as for irrigation pipe and where excessive pressures are encountered, a steel core is placed within the pipe and additional reinforcement is provided for, in the form of circumferential bars, steel fabric or spirally wound cages. Some of this special pipe has been used in this country in pipe lines where the working pressure is up to 195 lb. per sq. in.

Pipe for water supply is generally of the longer lengths, from 8 to 12 ft., to minimize joints. Rigid leakage requirements are

*Proceedings American Concrete Institute Vol. 25, p. 608.

complied with on final tests of the concrete pipe aqueducts. It is generally conceded that dense and impermeable concrete should be used for this service.

Force Mains. A number of sewer force mains have been built of reinforced concrete pipe, which are designed and constructed under the same principles as for water supply lines. The majority of sewerage systems are designed for gravity flows only but examinations of sewers indicate that in many cases, the lines operate under pressure during severe storms and particularly where sewers are overloaded.

The steel reinforcement of concrete pipe for gravity lines is designed for external loading and is adequate for resisting comparatively high internal hydrostatic pressures.

Subaqueous Construction. Where sewers are to be constructed under water for discharging sewage some distance from the shore to assure adequate dilution with the water or where intakes for water supply systems are similarly constructed, reinforced concrete pipe is extensively used.

Such pipe is generally built in long lengths, from 12 to 20 ft., and special joints are usually provided to simplify the operation of making joints under water, which is always very difficult. This construction requires special knowledge and experience and where the joints have been properly made, these lines have given first class service.

Since reinforced concrete pipe for this service is usually made with a dense and impermeable class of concrete and is cured and seasoned for an adequate period, it is an excellent type of construction for this work, particularly in sea water.

The steel reinforcement is usually placed with an adequate coverage of concrete so as to prevent corrosion of the steel. In Vancouver, B. C., in 1922, 66-in. reinforced concrete pipe was used in the construction of a large pier. The pipe was first sunk as caissons and later filled with monolithic concrete. The piers were examined by the writer in the fall of 1927 and were found to be in excellent physical condition.

CONDITION SURVEY

In 1929, more than 191 reinforced concrete pipe sewers were examined in 43 cities and large industrial plants in the United

States. The sewers varied from 24 to 108 in. internal diameter.

Sewers were examined and the data listed on printed forms by selected men representing the concrete pipe industry. In practically every case, an engineer or inspector representing the city or industrial plant accompanied these inspectors. In planning and carrying out the survey, a special effort was made to collect data thoroughly representative of service conditions.

These data have been tabulated and are being studied by a special committee which has under preparation standard specifications for reinforced concrete sewer pipe. Fourteen reinforced concrete pipe sewers were examined in the steel plants in the Pittsburgh district of Pennsylvania and six reinforced concrete pipe sewers in the steel plants of the Gary district of Indiana. One of these sewers had been in service since 1910, when the steel plant was built. Of the twenty sewers examined, which varied from 36 in. to 84 in. internal diameter, no concrete was found to have disintegrated.

Readers are referred to the JOURNAL *for November 1930 for discussion which may develop. Such discussion should reach the Secretary by Oct. 1, 1930.*

Fire Damage To and Repair of a Concrete Factory Building

BY JOHN G. AHLERS*

A FIVE story reinforced concrete factory was exposed March 3, 1929, to intense fire originating in an adjoining frame auditorium and amusement hall. The concrete building was occupied by the Bronx Drug Co., and used as a packing, manufacturing and distributing warehouse. The prevailing wind and draft created by the fire forced the flames and heat against the solid five-story concrete wall and drove around the corner of the building where it formed an eddy or blast, melting the wire glass and steel sash, letting the fire into the building and starting an intense combustion of the contents.

The contents of the fourth floor, where the fire entered, consisted of wooden packing cases, a large quantity of alcohol in bottles, inflammable chemicals and other highly combustible materials.

The hot blast, sucked around the corner of the building, also heated steel shutters on the lower three floors to a point where they were at a red heat, but the double protection of steel shutters and sash with wire glass prevented the fire from entering the lower floors. The fire was practically confined to the fourth floor, and the interior serious structural damage to the building was also mostly confined to this one floor, with some small damage to the fifth and third floors and minor destruction of the exterior concrete surface.

The fire was interesting to the concrete industry for the following reasons:

(1) The fire entered the building from the exterior by fusion of wire glass and steel sash.

(2) Damage to the structure was done by:
Spalling of concrete.
Overloading of heated slab.

*Secretary and Treasurer, Barney-Ahlers Construction Corporation, New York City.

Expansion of steel in concrete.
Fusion of glass and steel.

An estimate of the intensity of the heat can only be arrived at by judging the temperature necessary to fuse the glass and have it run down, taking the shape of icicles. This is estimated from data on melting point of glass, assuming a probable period of two

FIG. 1—VIEW TOWARDS CORNER FIFTH FLOOR; SASH BUCKLED AND WIRE GLASS CRACKED. THE BRIGHT SPOT IS THE POINT OF ENTRY OF FIRE ON THIS FLOOR

hours, to have reached the temperature of 550° C. The fire started between 2 and 3 a. m. and did not break into the concrete building until between 5 and 6 a. m.

The damage done to the exterior of the corner column by the flames and the blast, is indicated in Fig. 5. Likewise the spalling of the interior columns and the deflection and expansion of the slab are shown in Figs. 6 and 7.

The condition of the structure was such that the Fire Department pronounced it unsafe and the owners were instructed to make immediate repairs and take corrective measures.

REPAIRS SUGGESTED

The first natural method suggested for repairs was that of reinforcing the damaged columns and rebuilding the shattered

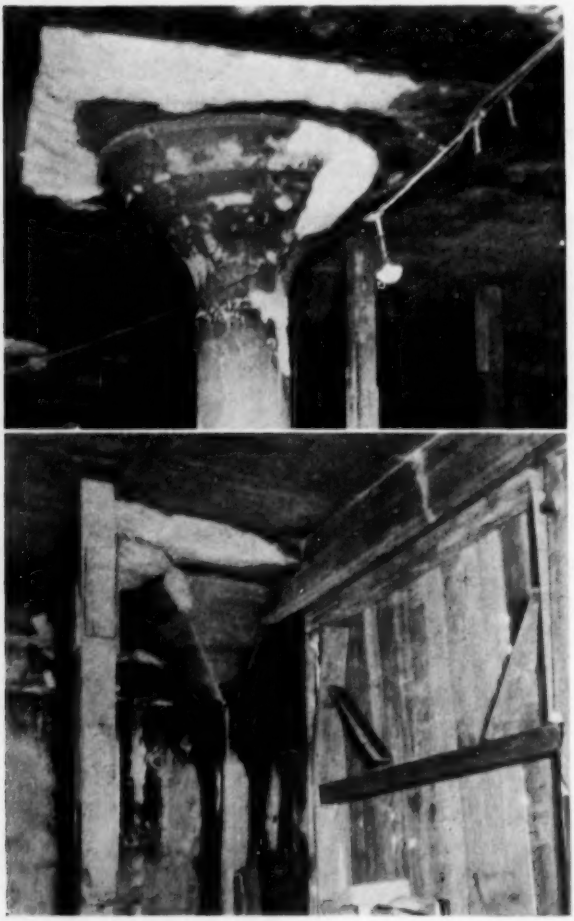

FIG. 2—SPALLED SECTION OF COLUMN 4, CEILING OF FOURTH FLOOR BEING FIFTH FLOOR STRUCTURAL SLAB. THIS IS ONE OF THE SECTIONS REMOVED. TEMPORARY SHORING IN PLACE. THE WHITE SPALLED AREAS SHOW WHERE CONCRETE DROPPED OFF DUE TO THE QUENCHING OF FIRE

FIG. 3—CORNER COLUMN UNDER ROOF. WALL TO LEFT OF COLUMN WAS BULGED TWO INCHES

floor slabs. As an alternate suggestion, however, it was recommended that the fifth floor slab be cut down for two and a half bays each way (as indicated in Fig. 4), the undamaged roof removed, and the structure rebuilt.

The owner naturally objected to reducing the head room around the columns, which would result from the construction of new supporting heads. There was also the possibility that the

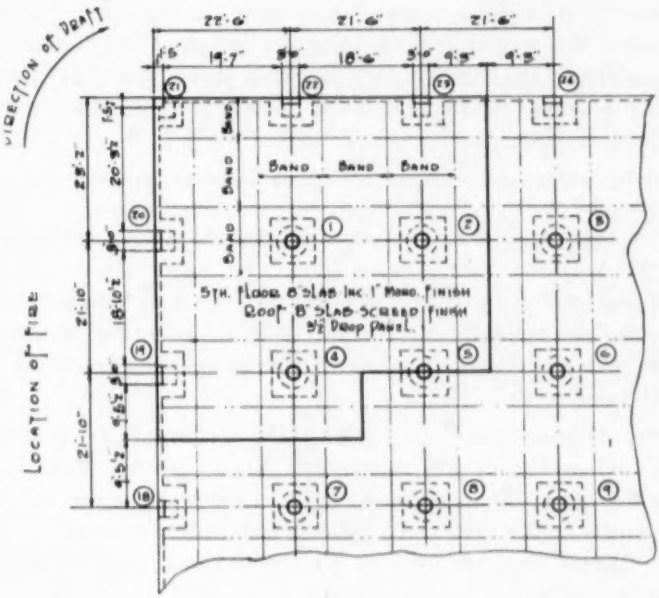

FIG. 4—LOCATION OF FIRE, DIRECTION OF DRAFT AND EXTENT OF DAMAGE TO FIFTH FLOOR

floors would not be allowed to carry as much as the structure had been designed for.

These facts were pointed out to the insurance company, and a comparative analysis was made as to the cost of (A) repairing and strengthening the structure as against (B) entire rebuilding of the damaged portion.

The estimated cost of (A) for the structural part alone, eliminating the question of various subcontract items that would be

the same in either case, was approximately $13,000. The estimated cost of (B) was $21,000.

After considerable discussion, and after rechecking of these estimates by consulting engineers, the owners were finally permitted by the insurance company to proceed on the basis of complete rebuilding of the damaged part of the structure, removing the roof immediately over it, to allow the wrecking of the damaged fifth floor.

Several interesting factors developed during the progress of wrecking the roof and the damaged portion of the fourth floor. It was found that the cost of wrecking and removal of the reinforced concrete work amounted to $14.00 per cubic yard of concrete in place, made up as follows:

Rental of compressor equipment, wrecking hammers, operators, etc., per cubic yard $6.70

Cost of construction of rubbish chute, mucking and disposal, per cubic yard $7.30

This cost of removal of the reinforced concrete was lower than expected and the information may be of value in similar instances where the anticipated cost of wrecking concrete may be considered excessive.

The structure was cut back to the center of the panels, as indicated in Fig. 4, and in several instances construction joints were followed. It was found that the concrete at the construction joints broke clean. In other words, there seemed to be no bond except that created by the reinforcing bars at the construction joints.

Fig. 4 is a plan showing the section removed. The roof over the portion indicated in the plan, was removed, even though not seriously damaged, to permit the removal of the fifth floor which was badly damaged in the panel between columns 1, 2, 4 and 5.

Design is two-way flat slab with bands as indicated. Concrete was cut to a straight line and steel left projecting six feet from this line for bond. Concrete at column was cut down to top of the capital. Exterior columns were removed down to and including capital in fourth story. Column 21 was badly damaged on exterior by heat, due to the draft around the corner as shown in Fig. 5. This column was also strengthened in the lower story.

Columns 1, 2 and 4 in the fourth story had a hollow ring and when wrecked, it appeared that the spiral reinforcement had expanded and loosened the 2-in. fireproof shell over about one-third the area of the column.

Fig. 5 shows damage to a corner column. This was the only column badly damaged on the exterior. Concrete was very badly pitted and disintegrated on the corner of the building

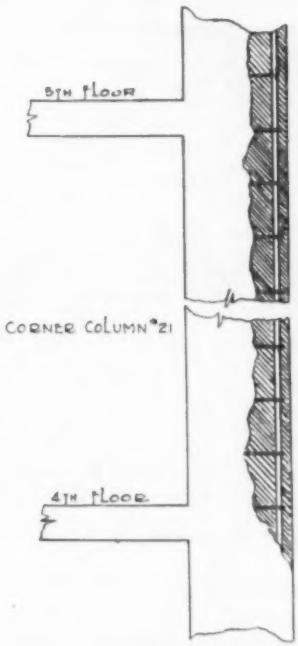

FIG. 5—SECTION OF CORNER COLUMN SHOWING EXPOSED REINFORCING STEEL

where the draft had drawn flames around and formed a very hot eddy, the same draft that melted the wire glass and sash on two floors and brought the fire into the building. Reinforcing steel was exposed as indicated for two and a half stories.

Exterior repairs consisted of cutting away to a minimum depth of 2 in. all disintegrated concrete, attaching wire reinforc-

ment to hoops and rebuilding column by pointing up to original line. The wire mesh consisted of chicken wire firmly tied to the exposed hoops and column bars. Wire reinforcement was attached to hoops and column rebuilt by pointing up to original line. Below fourth floor, column was reinforced by constructing

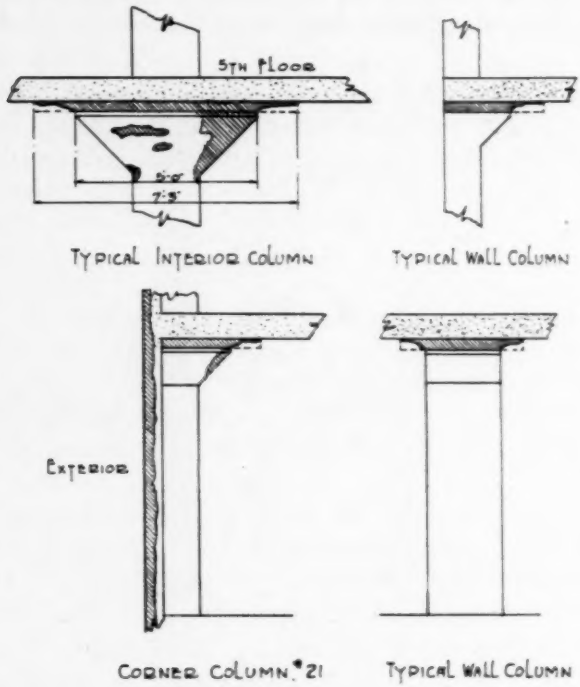

FIG. 6—SPALLING OF COLUMNS AND PANELS DUE TO INTENSE HEAT

an interior shaft filled through a hole in the fourth story. This shaft was tied into the existing column in this story.

Concrete on this exterior column was quite brittle and showed exposure to very high degree of heat, but there was not much indication of spalling as with the interior columns which had been subjected to jets of water from the fire hose.

Fig. 6 shows typical damage to column heads. The cross hatched sections indicate spalled areas.

Exterior wall from column 18 to 21 was badly spalled and pitted in fifth story. This wall which was sprung by the fire, was removed and rebuilt.

Columns 19, 20 and 21 had corners of dropheads spalled off, but shafts of the columns were undamaged. These columns had hoop reinforcement and no spirals. These hoop columns did not show any looseness in the concrete shaft, whereas the spiral columns 1, 2 and 4 had a very hollow ring and later the fireproofing outside of the spiral was found to have been badly damaged. All these columns were exposed to about the same degree of heat, which led to the conclusion that the spalling action on columns 1, 2 and 4 was caused by expansion of the spiral steel.

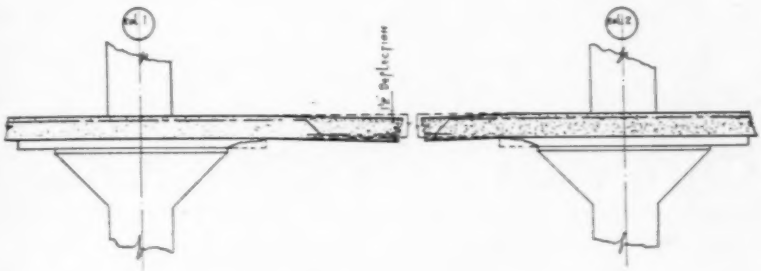

FIG. 7—DEFLECTION AND EXPANSION OF SLAB ON FIFTH FLOOR BETWEEN COLUMNS 1 AND 2

Decision to wreck the entire damaged portion was based upon the fact that structural steel members would be required to rebuild the dropheads if the same carrying capacity was to be guaranteed for the floor above.

In Fig. 7, a sketch shows a section through center of slab, fifth story, between columns 1, 2, 4 and 5. This was the only panel showing appreciable deflection and signs of disintegration. Examination of structure indicated that this sag was caused by the combination of three causes:

(1) Overloading by water soaked debris on fifth floor.

(2) Deformation of reinforcing steel either by flow under high temperature or expansion.

(3) Spalling and cracking, perhaps caused by item 2.

One of these columns was badly spalled, as shown in illustrations, and condition of exposed concrete indicated that this spalling took place after smoke and fire were partly or fully quenched, for the reason that the exposed spalled areas were clean and white and had not been sooted or discolored by smoke. It is possible, therefore, that this spalling action took place when the many streams of water were turned on by the Fire Department and the exterior of the concrete suddenly cooled.

This panel was so badly damaged and deflected, that it was necessary to have it removed. The destruction of this one panel influenced the decision on the remaining repair work and led to the ultimate removal of the entire corner of the building for the two structural slabs.

On the whole, it appears that the cost of wrecking and rebuilding entirely the damaged sections of the structure, as compared with making repairs and patching, was not out of line at all and that in similar instances recommendations should be made to remove the damaged portion entirely, rather than to make repairs which would decrease space and possibly reduce carrying capacity and rental value.

It would have been necessary to have removed all the fireproofing on the columns, outside of the spirals, and cast a new column around the spiral core. This column would also have required support by structural steel framework carrying the drophead of the slabs. In any case it would have been necessary to have removed the most severely cracked panels of the fifth floor as they showed a maximum deflection of 1½ in.

From the results of this fire and the cost of repairs the following conclusion can be drawn: Where reinforced concrete buildings are severely damaged, the damaged sections can be cut out easily and rebuilt at a nominal cost.

As a further attempt to draw lessons from this fire, a study was made of the increased cost of a concrete structure if limestone or trap rock aggregate were used instead of quartz. There were indications that the quartz pebbles had split and sheared from the influence of temperature change. It is certain that the fire

would have entered the building regardless of the type of aggregage used in the structural concrete. If trap rock or limestone aggregate had been used there might have been less spalling of column heads. It is likely that the columns would have spalled,

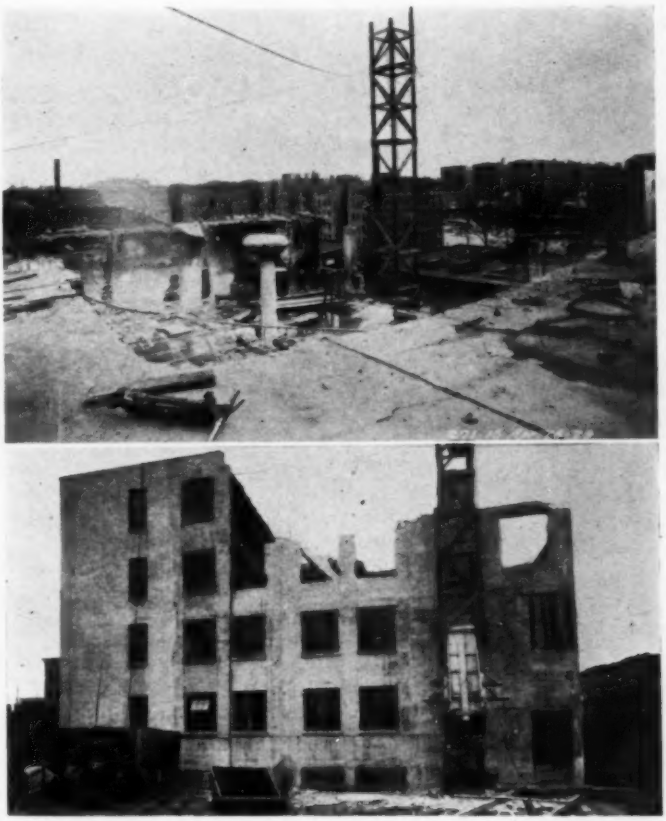

FIG. 8—ROOF SLAB WRECKED

FIG. 9—ROOF AND FIFTH FLOOR REMOVED

as the damage to the columns, apparently, was caused by the expansion of the spirals. It is certain, furthermore, that the concrete slab, being overloaded, the steel expanded by heat and

the under side of the slab exposed to fire, would have deflected to considerable extent and would have required repair.

The attached table is quoted by permission of the Manufacfacturers Mutual Fire Insurance Co., and applies, of course, only to buildings equipped with sprinklers. The building mentioned

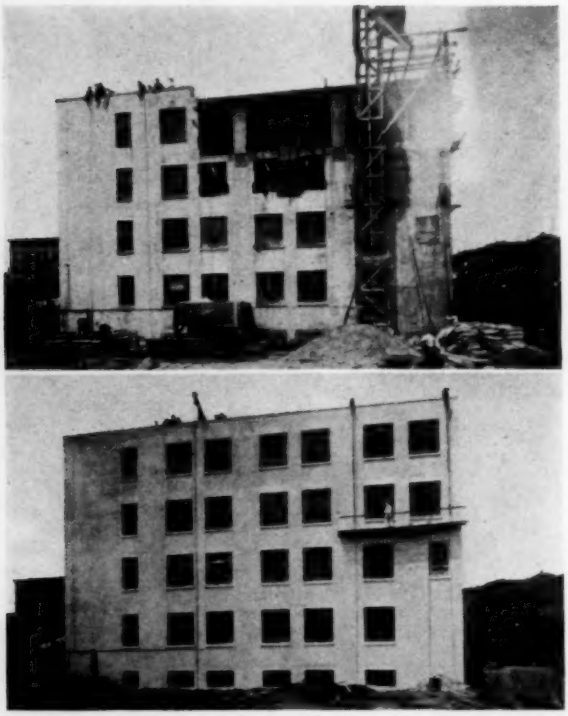

FIG. 10—STRUCTURAL FRAME WORK COMPLETED

FIG. 11—PUTTING ON THE FINISHING TOUCHES

in this report was not so equipped. This table shows, however, that the damage to the structure is merely nominal as compared to the damage to the contents. It is believed, therefore, that the conclusion to draw from this is that the requirement of the use of a broken stone aggregate in reinforced concrete would not be

justified, as the total increased cost resulting from the change in type of aggregate on all concrete structures in NewYork City for one year would be approximately $400,000, and the damage to all concrete structures by fire is only a fraction of this amount.

The insurance company carrying the risk on this structure asserted that no structural steel framework, even though "fire-proofed" could have withstood exposure to this fire without entirely collapsing over the fourth story, possibly pulling down the roof structure so that at least the two upper floors would have been destroyed entirely and would have overloaded the lower floors.

It was estimated that the loading on the fifth floor at the height of the fire, due to the saturated materials and the water load, amounted to not less than 200 lbs. per sq. ft.

TABLE I

	Building	Contents	Total	No. of Fires	Average Building Loss Per Fire	Per Cent Building Loss
Reinforced Conc.	$20,682	$448,710	$469,392	151	$137	4.4
Plank on Timber	25,938	130,201	156,139	107	242	16.6
Boards on Joists	49,065	184,755	233,820	65	756	21.0

Readers are referred to the JOURNAL *for November 1930, for discussion which may develop. Such discussion should reach the Secretary by Oct. 1, 1930.*

Recommended Practices in the Use of Cast Stone*

Report of Committee 704, Cast Stone Standards

LOUIS A. FALCO, CHAIRMAN†

Cast stone should be composed of portland cement and hard, durable aggregates of such colors and in such proportions as to produce, when finished, the desired appearance, strength and density. All materials used in the manufacture of cast stone should pass the standard tests for such materials as adopted by the American Society for Testing Materials.

Cast stone should be manufactured in an inclosed building and after being removed from molds should be stacked for curing under moist conditions for at least seven days. It should then be allowed to dry out, being protected from air currents and the sun's rays. Generally, cast stone should be aged at least 14 days before being delivered to the job.

Cast stone should be the product of a manufacturer having the capacity and facilities for furnishing the quality, sizes and quantities of cast stone required without delaying the progress of the work and whose products have been previously used and exposed to the weather with satisfactory results.

Cast stone should have a compressive strength of at least 5,000 lbs. per sq. in. and an absorption of not more than 7 per cent nor less than 3 per cent when tested in accordance with the American Concrete Institute tentative specification for cast stone (P-3-A-29T).‡

Cast stone should be definitely specified according to the following classification:

*Presented at the 26th annual convention, Feb. 1930, approved by a majority of the committee consisting, beside the chairman, of: C. G. Walker, secretary, P. H. Bates, George J. Eyrich, Herman Frauenfelder, J. W. Lowell, Duncan Thomson.

†President, Decorative Stone Co., New Haven, Conn.

‡Proceedings of the Amer. Conc. Inst., Vol. 25, 1929, p. 550.

Cut Cast Stone:

Planer rubbed finish
Bush hammered finish
Machine tooled finish
Crandalled finish
Hand tooled finish
Sawed finish

Surfaced Cast Stone:

Hand rubbed finish
Brushed finish
Acid washed finish

Plain Cast Stone:

The architect or purchaser of cast stone should indicate the approximate color and texture desired in the cast stone and each manufacturer bidding under the specification should submit samples showing the quality and finish of the cast stone which he proposes to furnish.

Cast stone should conform in every particular to the sample as finally selected and approved by the architect or purchaser.

The architect or purchaser should have the privilege of selecting pieces of cast stone at random for testing purposes. Tests should be made by a laboratory approved by both the architect or purchaser and the manufacturer and should be paid for by the manufacturer.

Scale and detail drawings should be furnished by the architect wherever necessary.

Clay or plaster models of all ornament should be provided by the cast stone manufacturer and should be approved by the architect either from photographs or by personal inspection before any work is executed therefrom. Molded ornament should faithfully reproduce the character and feeling of the drawings and models.

Based upon the drawings furnished by the architect, the manufacturer should prepare complete shop drawings and setting diagrams showing jointing, bonding, and anchoring for all cast stone work. The dimensions and setting number of each piece of cast stone should be clearly indicated on the shop drawings.

The general contractor should furnish to the cast stone manufacturer the information necessary to enable him to check the

cast stone properly with connecting structural work to insure that all these items will be in perfect agreement.

Copies of all shop drawings should be submitted to the architect, one set to be approved by the architect in writing thereon and returned to the manufacturer before the manufacture of cast stone is started.

Cast stone should be reinforced as necessary to withstand all stresses to which it may be subjected in transit and in place. Reinforcing should not be placed within 1½ in. of the surface.

Lifting rings or Lewis holes should be provided in all cast stone where necessary for handling or hoisting purposes. Holes and sinkages should likewise be provided for all anchors, dowels, cramps, etc.

Anchors, dowels, cramps, etc., should be wrought iron, zinc coated or dipped in asphaltum or red lead after fabrication. In general not less than two anchors should be provided for each piece of built-in cast stone 4 ft. or more in length and in no event should there be less than one anchor to each piece of cast stone.

Beds and joints should be full and square for the full depth of the piece and be scored to provide a bond for the mortar.

All exposed surfaces of cast stone should be true, free from projections or depressions, and faces in the same plane should be flush at the joints. Moldings and arrises should be sharp, true and continuous at the joints.

Joints should be plumb and level and of uniform width. At least every third course of ashlar should bond not less than 4 in. into the backing, and no piece should have less than 4-in. bed. Reveals of openings less than 12 in. deep should be without vertical joints. Pieces at grade should extend not less than 4 in. below grade. Pieces that are to be bonded with other material should be kept not less than 4 in. from the back of the wall, and the bed joints of pieces bonding with other masonry facing material should coincide with the horizontal joints of the connecting material.

All projecting courses such as window sills, belt courses, copings, cornices, etc., should have drips under the projecting edge. Pieces with exposed top surfaces should have a wash.

Reglets, rabbets, slots, etc., should be provided in cast stone to

receive all flashing, sheet metal work, or other work which comes in conjunction therewith.

Pieces coming in contact with structural steel or fireproofing should be shaped as required to fit connecting work.

Cast stone should be carefully packed for shipment, using every reasonable precaution against damage in transit. When being removed from cars or trucks, cast stone should be handled throughout by competent workmen and by such methods as will guard against soiling, mutilation or chipping. Cast stone should be stored at the building site in such a manner that it rests entirely clear of the ground and should be protected by proper means from damage to arrises and from contact with anything that would result in damage from the accumulation of dirt, dust, grease or other staining or disfiguring agents. During extended periods of storage at the building site, cast stone should be covered with tarpaulins or stout non-staining paper. In no case should the cast stone be stored under trees or other overhanging objects.

Cast stone should be set in mortar composed of one volume portland cement, one volume cold lime putty and four volumes clean, well graded sand, or mortar of equal quality. Lime putty should be made from the best quality lump lime slaked with cold water, screened through a $\frac{3}{16}$-in. mesh screen into a settling box, allowed to stand not less than one week and then mixed with sand and stacked to age. The correct amount of cement should be thoroughly worked into small batches of the mixture just prior to its use for setting cast stone.

All exposed surfaces of cast stone should be cleaned and the mortar surfaces of each piece should be sponged with clean water just before setting. Each piece should be set accurately, true to line, level and plumb by competent stone setters, with full flushed joints, filling all anchor holes. The exposed faces of cast stone should be kept free from mortar at all times.

Pieces of cast stone in heavy projecting courses and other heavy projecting members should be propped up until the anchoring has been built in and sufficient work is in place above to hold such members securely in position. Heavy blocks of cast stone should not be set until the mortar in the joints below has

thoroughly set. The use of pinch bars except on the backs of cast stone should be prohibited.

Each piece of cast stone, except where otherwise specified should rest on a full bed of mortar in sufficient amount to fill out to the edges of the piece on all sides. The use of wooden wedges should not be permitted. Each piece of cast stone should be adjusted on its bed by striking with a wooden mallet or ram. Any piece which does not find its proper bed and solid bearing should be lifted and reset on a new bed of mortar. All anchors and dowels as provided for should be set in position as the courses are laid. All holes and sinkages around anchors, dowels, cramps, etc., should be entirely filled with setting mortar. Joints should be raked out ¾ in. deep on the face for pointing.

Only the ends of lug sills should be set in a full bed of mortar, balance of joint to be left open until pointed.

All cornices, copings and projecting belt courses and pieces forming gutters should be set with the vertical joints dry. These joints should be caulked on the exterior profile with oakum and should then be filled to within ¾ in. of the top with liquid grout composed of one volume cement to one volume of fine sand, prehydrated for at least three hours and of as thick consistency as can be poured into the joints. After the grout has thoroughly set the top joint should be filled with lead wool caulked tightly against the edges and left slightly convex on the top. Oakum should be removed and face joints pointed as hereinafter recommended.

After being set all cast stone work should be properly protected from damage by means of boards or other suitable covering where necessary until completion of the work.

When ready to be pointed the face of all cast stone should be scrubbed with a fiber brush using soap powder and water and should then be thoroughly rinsed with clean water. Mortar splashed or dropped on the face of cast stone during setting should be immediately washed off with a sponge and clean water.

Face joints should be brushed out clean to a depth of ¾ in., the joint lightly dampened and then carefully pointed. Pointing mortar should be composed of one volume portland cement, not more than 1¼-volume cold lime putty, and two volumes of clean,

sharp and well graded sand. Pointing mortar should be premixed in small batches and allowed to stand three hours before being used. Pointing mortar should be packed solidly into all joints. Pointing should not be done in freezing weather or in locations exposed to hot sunshine unless properly protected.

Readers are referred to the JOURNAL *for November 1930, for discussion which may develop. Such discussion should reach the Secretary by Oct. 1, 1930.*

Discussion of Paper by Norman M. Stineman

Moment and Shear Diagrams for Continuous Beams and Rigid Building Frames*

BY ARTHUR R. LORD†

Since I was afforded the privilege of examining this paper and of offering suggestions, all of which were cheerfully accepted, my criticism must be as much of myself as of the author. We both overlooked an essential matter, in expressing the stiffness factor of columns and girders in terms of I/H and I/L solely, instead of in terms of EI/H and EI/L. The effect of the elastic property of the material on stiffness is so universally known that the correction would be made where necessary, probably, without this note. But it is, perhaps, worth while as a protective measure, to state that E_c does not always cancel out; that the proper value of E_c must be used when the members meeting at a joint are composed of concrete of markedly different properties. It is fairly common to use several qualities of concrete in a tier of columns, and in such cases, whenever one member at the joint is made of different concrete from the others, the relative stiffness must be expressed in terms of EI/H and EI/L.

BY U. T. BERG‡

The writer would suggest that if Mr. Stineman's diagrams are to be issued as standard practice of design, that they should be revised as far as the values for negative moments are concerned. The intermediate values between support and O are all unnecessarily severe.

The negative moment for any span AB is made up of the following 3 conditions of loading:

(a) The maximum values near support B occur when AB is fully loaded. Span adjacent to B fully loaded, all other spans

*Journal of the American Concrete Institute, January 1930, *Proceedings*, Vol. 26, p. 211.
†Consulting Engineer, Chicago.
‡Structural Engineer, New York City.

alternately loaded and unloaded. See A on sketch.

(b) The maximum values nearer middle part of span. AB unloaded, other spans alternately loaded and unloaded. See B on sketch.

(c) The maximum values between 0.1 L and 0.2 L from support B occur when span AB is fully loaded from A to about 0.8 L. Other spans alternately loaded and unloaded. See C on sketch.

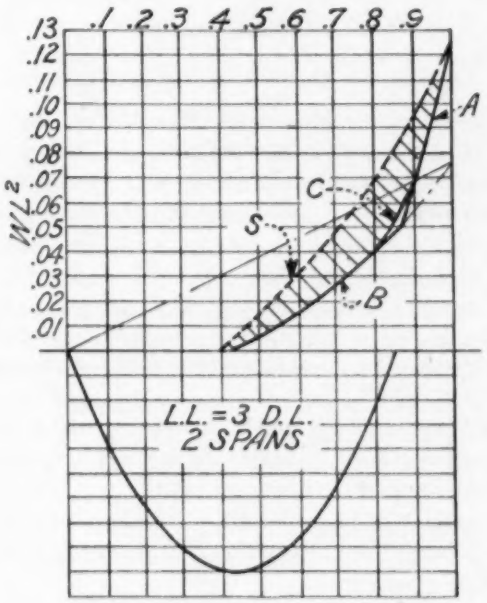

FIG. 1

The values C compared with B and A are only at the most about 7 per cent shy in this region and for all practical purposes it is correct to round off the intersection of curves A and B for a length of 0.05 L on each side of intersection.

Curve S represents Mr. Stineman's negative moment for the same conditions of spans and relation of live load to dead load and hatched area represents the unnecessary increase beyond the largest possible moments.

While the example is taken for an end span, it also applies to other spans in a general way.

Discussion of the paper by Arthur R. Lord:

"Design and Cost Data for the 1928 Joint Standard Building Code"*

BY I. OESTERBLOM†

Design of Compound Beam Sections

In the discussion by Messrs. Hadley and Way of Mr. Lord's "Design and Cost Data"—(See *Proceedings* 1929, p. 828)—there is a statement which needs repeating, both for the benefit of the people engaged in the art of making concrete structures and for the teachers in the schools, where this noble art is taught. Very truly and significantly they say:

> The apparently involved and forbidding calculations which its use entails have unquestionably led to the rejection of concrete and the use of other materials in numerous cases, despite the adaptability and even economy which reinforced concrete offers.

In the early years of the newborn art, attention was specially directed toward methods to figure the elementary as well as compound sections correctly. Later on, as the need to recognize the fact of continuity became apparent, even more thought was spent on the development of reasonable formulas and methods for correct bending moments.

Later developments, a more complete study of the problems involved, and especially analysis for elimination of secondary or insignificant causes have given birth to a great deal of simplification and at the same time greater accuracy.

Old methods and formulas, however, due to the tremendous inertia of years of propaganda and service are hard to down. It is to be hoped therefore that the contribution from Messrs. Hadley and Way pointing out simpler ways and also what others

*A. C. I. *Proceedings*, Vol. 24, 1928, p. 537. It is perhaps more accurate to designate this contribution from Mr. Oesterblom as discussion of discussion—based, as it is, on discussion of Mr. Lord's paper by H. M. Hadley and W. F. Way. *Proceedings*, Vol. 25, 1929, p. 829.—Editor

†Chicago, Ill.

have done to make the mathematics of design less cumbersome, will receive the attention it deserves.

The theory of the simplified method proposed is well explained on pp. 830-832; the examples and tables on subsequent pages are also excellent. All designers should read these and, no doubt they will thenceforth forget the old, yard long, several story high formulas from the text books, and use the simplified method only. Not only is it much faster; it is also much clearer and puts a real meaning into the old formulas and thus justifies their elimination. (The above is said with special compliments to the teaching profession.)

While I thus take the opportunity to stress the usefulness of the simplified method, I wish to point out that it was published by the writer clearly, completely, and covering all kinds of combinations—even steel in compression—as far back as March 1910.

At that time reinforced concrete had made some definite strides towards popularity—although many architects still insisted that concrete "had no business except below ground"—but good books and reliable information were still scarce.

The writer at that time was interested in power house construction and studied reinforced concrete from German books. March 19, he went before The American Society of Swedish Engineers, Brooklyn, N. Y., with an elementary paper on "Design of Reinforced Concrete Beams."

I beg to quote from the published proceedings of the Society as follows: (The notations are those at that time in use in German books; no American notations were standardized or universally adopted.)

REINFORCEMENT IN COMPRESSION (P. 8)

The introduction of reinforcement both in the tension and the compression zones for such cases, where it is necessary to place a strong beam in a relatively small space, has given rise to a number of formulae and methods of solution, more or less elaborate, but for practical purposes unsatisfactory. The action inside of the beam being well understood, it is possible to use all the working formulae for beams with reinforcement in tension only and corresponding tables, and to apply adjustments after preliminary results have been first obtained.

The resisting moment of the beam can be considered as consisting of two parts, $M_1 = M + M_2$ .....(21)
of which M is the moment produced by the concrete in compression and that part of the steel in tension which counterbalances the concrete in compres-

sion, and M_2 is the moment produced by the steel in compression and the remainder of the steel in tension.

With the depth of the beam given by the limited space conditions M is first solved by the use of equation (9). M_1 is determined by the load conditions and additional reinforcement must now be provided to take care of M_2 which is now know to be equal to $M_1 - M$.

If y_2 is the lever arm between the additional steel in tension and the steel in compression, we find the total tension or compression in the added steel to be $\frac{M_2}{y_2}$ and the required area in tension $\triangle_1 = \frac{M_2}{y_2 t_o}$ .....(22)

The requirements for elasticity in the beam make it essential that in the compression zone the steel stresses should be n times the concrete stresses at the point where the steel is placed. If therefore a steel area is introduced, which does not violate this rule, the addition of extra reinforcement will not affect the depth of the compression zone or the length of the lever arm for the moment M. The concrete stresses at the point of reinforcement are $\frac{x - (h - y_2)}{x} c_o$ and the available steel stresses therefore, after deduction has been made for the replaced concrete, $\frac{(n-1)[x-(h-y_2)]}{x} c_o$. The required steel area in compression is therefore $\triangle_2 = \frac{M_2 X}{(n-1)[x-(h-y_2)]\, y_2 c_o}$ .....(23)

T-BEAMS (P. 10)

Besides the rectangular beam * * * there is another form of beam with a compound rectangular shape extensively in use, the so-called T-beam. * * * Numerous formulae have been deducted governing the conditions in a T-beam and all of them, if correct are quite complicated and cumbersome. * * *

As a matter of economy in the use of building materials, it is frequently desirable to increase the depth and therewith the lever arm, which will reduce the total stresses and lower the amount of concrete in the flange and steel in the web.

There are, of course, an unlimited number of possibilities, all of which will supply the proper moment of resistance to the exterior bending moment, and all of which will also show proper material stresses, provided the T-beam is well proportioned and the correct amount of reinforcement is used. One can therefore select any reasonable depth, dependent on local conditions. If a solid rectangular beam were designed with this depth h and a width b, its moment of resistance, according to equation (9) would be $M = \frac{bh_2}{c_1^2}$ ....(26) which is in excess of the required moment. The required resisting moment being M_p we have $M - M_p = M_n$.....(27) where M_N represents the moment of that part of the full rectangular beam

which should be deducted from the full moment to give the desired moment. This negative moment is made up of certain steel stresses, taken up by that part of the reinforcement for the full rectangular beam, which is in excess of the requirements in the T-beam or corresponding concrete stresses, located below the lower flange line and a lever arm which is $h - x + \frac{2x_n}{3}$, if x_N is the compression depth below the lower flange line. Call the concrete stresses at the lower flange line c_n and the steel area, which takes the excess tension $\triangle_n$. We then have

$$\frac{M_n}{h - x + \frac{2x_n}{3}} = \triangle_n t_o \quad \ldots\ldots (28)$$

$$\frac{M_n}{h - x + \frac{2x_n}{3}} = \frac{bx_n c_n}{2} \quad \ldots\ldots (29)$$

* * * The reduction of steel from that required in the full rectangular beam follows directly from equation (28)..

(CONCLUSION PAGE 16)

* * * Of the new material in this paper I wish to call attention to my method of solving double reinforced and T-beams. It is a new method for an old theory and should therefore invite scrutiny and discussion.

As one grows older and has to battle with the real problems of engineering one naturally develops a dislike for useless formulas and particularly those which have a tendency to conceal the simple realities. At that time, being new from school, I expressed many of my ideas in formulas, which now I would express by simple common rules or proportions. They are none the less a definite expression of the simplified methods later on introduced by Messrs. Beard, Hadley and Way.

Subsequently as a member of the engineering staff of the Truscon Steel Co., I had ample opportunity to apply my simplified method. A T-beam chart was issued, also examples to illustrate. The chart is dated 4-16-13 and is a copy of personal chart of previous date. The examples, second edition, were issued 12-23-14.

The first example shows a T-beam with two thicknesses of slab the second a T-beam with the slabs on either side at two different heights and with two different thicknesses. Both examples are solved by the simplified method *correctly* in two or three minutes, whereas by the usual elaborate formulas, hours of time would be

required. Both sections are unusually complicated from the formula point of view—even with the addition of compression steel they would be easy to figure by the simplified method.

The second of the above two examples is identically the same in character as the second in Messrs. Hadley and Way's discussion.

Later on I carried my consultation practice to several Asiatic countries. In China I found fertile ground among the many very brilliant young Chinese engineers, who gratefully accepted my method and have used it since the year 1915. In matters of this kind many Asiatics are more receptive than the Americans—perhaps principally because all modern engineering is new to them and the inertia of old methods have no retarding influence.

More recently I have used my method on more complicated problems, viz., two different kinds of concrete in compound reinforced beams.

This is an entirely new development in concrete engineering and I know that with the usual formulas properly extended to take care of a third material the task would be hopeless.

My recent studies are far from being purely academic. It has to deal with the very specific and completely practical problems of dead weight reduction for tall buildings. For the purpose two kinds of concrete have to be used in combination; one concrete with an expanded aggregate and another an expanded concrete. Both of these materials are light in weight, but have otherwise different but supplementary properties. They combine for reinforced units of unusual strength and extremely low weight and they present at the same time a most interesting mathematical problem.

In view of all the above, I wish to say with Messrs. Hadley and Way: Let us drop the old cumbersome formulas and let us pray that all students may rebel against the teaching of them in school, when simple methods are available, ever so much clearer and more instructive.

BY H. M. HADLEY* AND W. F. WAY*

IT IS interesting to learn that as long ago as 1910 Mr. Oesterblom presented in his paper before the American Society of

*Seattle.

Swedish Engineers methods of design of reinforced concrete beams closely resembling those set forth in our discussion. Without a full statement of the notation being given, it is difficult to follow and check Mr. Oesterblom's formulas. The published proceedings of The American Society of Swedish Engineers are not, so far as we know, available for reference in Seattle; neither are they listed in the Engineering Index of 1910.

Regardless of this fact, Mr. Oesterblom's statement shows the same basic conception of the beam reinforced for compression as we presented with acknowledgment to Mr. Robt. S. Beard and a conception of the T-beam as the difference of two rectangular beams similar to ours. We cannot see, however, how Mr. Oesterblom's equation (27), all the terms of which are clearly stated and defined, can in general be true. The moment capacity of the rectangular section which is deducted from the gross overall rectangular section is wholly independent of the external moment to which the beam is subjected, whereas Mr. Oesterblom's formula apparently makes it a function of the external moment. Such treatment would lead to incorrect arithmetical consequences.

Also it is to be observed that values of K and P are necessary to make the simplified design method a practical reality.

BY I. OESTERBLOM

THE brevity of the extract may account for the misunderstanding of equation (*27*). Not containing any dimensions in reference to either steel or concrete it does not express the properties of the section. It merely expresses in symbols the method of procedure. It seems to be identical, except in form, with the basic idea of Messrs. Hadley and Way to segregate the two elements of resisting force couples and thereby simplify calculations of resisting properties.

To say it in words instead of symbols we have the very simple and obvious truth that if an elementary rectangular concrete section, reinforced to the limit, is not strong enough to resist the external bending moment, an additional element of strength must be provided, or if it is too strong, elements of strength may be deducted.

This is all the equation endeavors to say.

It is the second part of the problem to find what this second element of strength should consist of and how it should be proportioned. There may be added projecting slabsections of various thicknesses and positions with steel to balance in elastic equilibrium; or additional steel on both sides of the neutral axis may be necessary or preferable. Fortunately, through the segregation, the figuring of the additional elements, no matter how many or how complex, is exceedingly simple.

That this can be done by the intelligent use of the well known "*p*"*s* and "*k*"*s*, values of which can be found in every reference book on the subject and that no special formulas or tables need be devised, should be considered a point of advantage rather than the reverse.

More recently an article in *Concrete* has been running serially employing the same principle of segregation; it is to be hoped that soon the absurdly complicated and most unnecessary formulas and tables of doubly reinforced and T-beams will disappear entirely from our text books.

Discussion of Paper by Hardy Cross

Design Of Reinforced Concrete Columns Subject To Flexure*

BY N. H. ROY† AND F. E. RICHART‡

The following study of the modulus of elasticity of concrete and its bearing on design, is the result of many discussions that the writers have had with Professor Cross and much of the work was done by Mr. Roy in connection with graduate courses in reinforced concrete, taken simultaneously with Professors Cross and Richart. The results given are therefore quite familiar to Professor Cross and he has encouraged their publication as supplementary evidence relating to certain phases of his paper on the action of columns subjected to bending. The discussion is as much a discussion of the Joint Building Code of the Institute as it is of the paper, and the latter was probably written with the purpose of drawing more critical attention to certain of the code provisions.

In his paper, Professor Cross points out that in certain forms of continuous reinforced concrete framing, in which the moment induced in a column is a function of its stiffness, an increase in the strength of the concrete used may be of no value and may even be a detriment to the load capacity of the column. This conclusion is based upon the assumption that the modulus of elasticity E_1, is directly proportional to the compressive strength f'_c.

The modulus of elasticity has been expressed as a function of compressive strength in various ways. In the reports of the 1916 and 1924 Joint Committees[1,2] values of E for design purposes were given for three classes of concrete based on 28-day compressive strengths as follows:

*Journal of the American Concrete Institute, December 1929; discussion, April 1930.
†Research Assistant, Theo. and Appl. Mech., University of Illinois.
‡Research Assoc. Prof., Theo. and Appl. Mech., University of Illinois.
1, 2, et. seq. see numbered list of references at end of this discussion.

For f_c' over 1500 and under 2200 lb. per sq. in., E = 2,000,000 lb. per sq. in.

For f_c' over 2200 and under 2900 lb. per sq. in., E = 2,500,000 lb. per sq. in.

For f_c' over 2900 lb. per sq. in, E = 3,000,000 lb. per sq. in.

The Joint Building Code[3] of the American Concrete Institute, 1928, specifies E for design purposes as 1000 f_c', where f_c' is the 28-day compressive strength of the concrete.

While undoubtedly these relations between E and f_c' are based upon the results of many tests, the aim of this discussion is to present test data which may indicate limits of application and the effect of certain variables upon the accuracy of the general relations.

The value of E enters two general design problems, (1) The distribution of stress between steel and concrete in a reinforced beam, slab, or column, and (2) the distribution of bending moment between members of a rigidly connected frame. The first case is not of great importance, since a variation in the value of E used will produce only a minor difference in the load carrying capacity of the member. The use of an incorrect value of E in the problem of distributing moments in a continuous structure is more serious. As indicated by Prof. Cross, if a small column frames into a deep heavy girder, the amount of moment thrown into the column is almost directly proportional to its value of EI (where I represents the moments of inertia of the section). A 10 per cent change in EI will produce a similar change in the induced moment. Experience shows that such a column may have high stresses and any attempt of the designer to relieve them will run into difficulty. An increase in section or in reinforcement will obviously cause an even greater increase in I. If E is equal to 1000 f_c', the use of a stronger concrete will likewise increase E, producing, in a plain column, a proportional increase in moment. In a reinforced concrete column, the increase is similar, though not so simply generalized in statement. If, on the other hand, E is constant for the stronger concretes, (say for f_c' of 2900 lb. per sq. in. and above, as given by the Joint Committee Report) then the use of stronger concretes would be quite effective in providing for column moments in buildings and rigid frames.

FACTORS AFFECTING MODULUS OF ELASTICITY

It seems logical that the modulus of elasticity for concrete should depend upon the modulus of elasticity of its constituent materials, aggregates and neat cement matrix. Some average data[4] are given in Table 1 on the modulus of elasticity of various kinds of granite, limestone and sandstone similar to those used in concrete. Sand and gravel may be expected to have values corresponding to those of the "mother" rock. Values of the modulus of elasticity of neat cement, from tests of 6 by 12-in. cylinders, of Universal cement, 28 days old, are given in Table 2[5]. These neat cement cylinders, ranging from very wet to very dry mixes, gave values of E relatively low with respect to f'_c. Apparently the cement paste is not the element in a concrete mixture that produces a high value of E. On the other hand, suitable aggregates might be expected to produce a high value of E. It is recognized that the stiffness of a small portion of cement binder restrained by the surrounding mass of concrete may be greater than that of the 6 by 12-in. cylinder but the same thing is true of the aggregates, so that the relative values given in Tables 1 and 2 should apply to the constituents of the concrete in place.

The data of Table 2 show that while an increase in water-cement ratio should decrease the value of E considerably, the change is far less, relatively, than that produced in f'_c. Thus while a change in water-cement ratio from 0.32 to 0.76 reduces E to about half the original value, it reduces f'_c to one-fourth of its corresponding value.

TABLE 1. PROPERTIES OF VARIOUS KINDS OF STONE

Summarized from Tests of Metals, Watertown Arsenal, 1890, 1894, 1895. Many of these tests show increasing modulus of elasticity with increasing stress

Kind of Stone	Compressive Strength Thousands of Lb./Sq. In.	Modulus of Elasticity in Compression at Low Stresses Millions of Lb. Per Sq. In.
Granite	9.0 to 29.0	4.5 to 9.0
Limestone	4.0 to 21.0	3.0 to 14.0
Sandstone	5.0 to 19.0	1.0 to 4.0

TABLE 2. INITIAL MODULUS OF ELASTICITY FOR NEAT CEMENT

From tests made at University of Illinois, 1919. 6 by 12-in. compression cylinders. Age 28 days, moist room curing. Universal Portland Cement

Mixing Water		f'_c	E	Ratio $\frac{E}{f'_c}$	Unit Shortening at Max. Load
Per Cent By Weight	Water-Cement Ratio By Vol.	Comp. Strength Lb. Per Sq. In.	Initial Modulus of Elasticity Lb. Per Sq. In.		
19	0.29	1850	2,250,000	1210	.0013
21	0.32	8390	3,520,000	420	.0032
23	0.35	8245	3,250,000	394	.0034
25	0.38	7865	3,080,000	392	.0036
27	0.41	7110	2,820,000	396	.0033
29	0.44	6505	2,700,000	415	.0034
31	0.47	5995	2,540,000	424	.0034
33	0.50	5480	2,425,000	443	.0031
35	0.53	5025	2,305,000	458	.0031
37	0.56	4075	1,960,000	481	.0028
39	0.59	3600	1,900,000	528	.0029
41	0.62	3570	1,765,000	494	.0028
43	0.65	3385	1,730,000	511	.0029
45	0.68	3195	1,730,000	542	.0026
47	0.71	2930	1,595,000	545	.0021
50	0.76	2015	1,655,000	822	.0014

The foregoing indicates the improbability that a simple proportion can be found to express the relation between E and f'_c for concretes in general. The relation undoubtedly depends upon richness of mix, consistency used, kind and quality of aggregate, gradation of aggregate, age of concrete, moisture content of concrete, and other factors.

VALUES OF E FROM TESTS OF CONCRETE

In referring to the data of various tests of the modulus of elasticity of concrete it must be expected that some variation in precision of strain measurement as well as of control of mixing and curing of the concrete, will be encountered. However, it seems likely that in most cases general trends of the test values will be properly shown, even though great accuracy is not attained. There is no need for values of E to four significant figures, but it is important to have values correct to two significant figures.

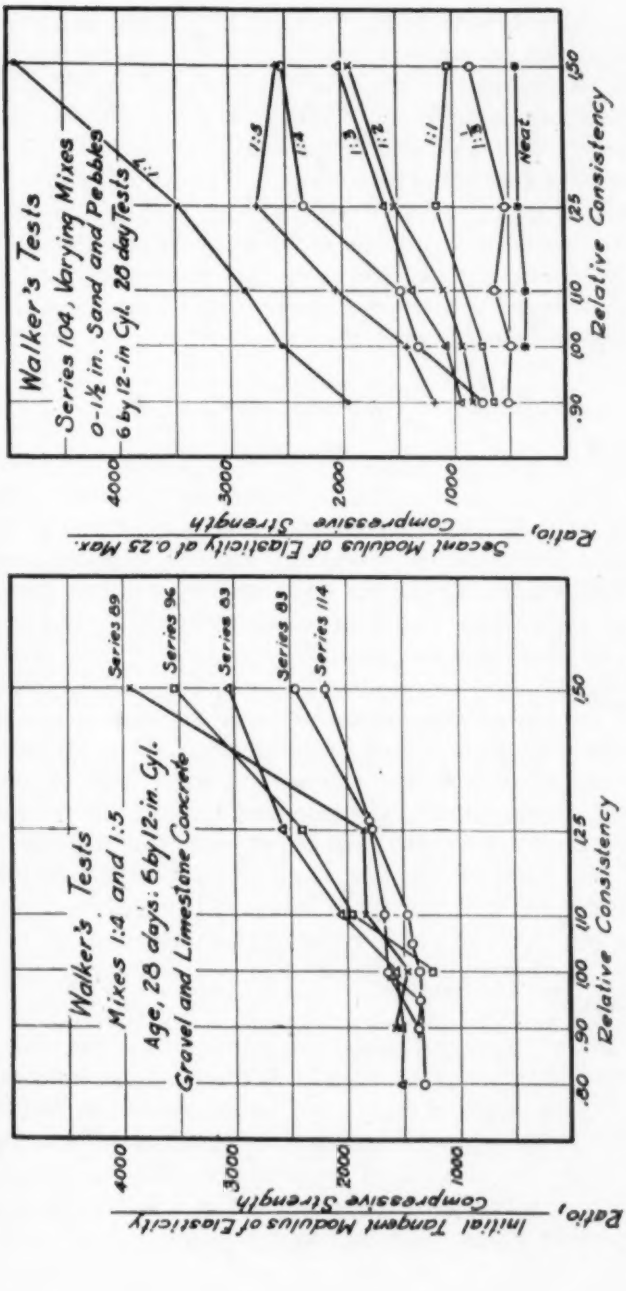
Walker's Tests
Mixes 1:4 and 1:5
Age, 28 days: 6 by 12-in. Cyl.
Gravel and Limestone Concrete
Series 89
Series 96
Series 83
Series 83
Series 114
Ratio, Initial Tangent Modulus of Elasticity / Compressive Strength
Relative Consistency
(a)
Walker's Tests
Series 104, Varying Mixes
0-1½ in. Sand and Pebbles
6 by 12-in Cyl. 28 day Tests
1:5
1:4
1:3
1:2
1:1
½:1
Neat
Ratio, Secant Modulus of Elasticity at 0.25 Max. / Compressive Strength
Relative Consistency
(b)

FIG. 1

Most tests to determine the modulus of elasticity in compression have shown the stress-strain curve to be a straight line up to a load of one-fourth, or more, of the ultimate strength. One exception is seen in the tests by Walker[6], in which some curvature was reported beginning with initial loads. Walker's initial tangent moduli will generally be found to be somewhat higher than initial moduli found by other investigations, and it seems likely that Walker's secant modulus at 25 per cent of ultimate load (which he shows will average about 82 per cent of his initial modulus) is more nearly comparable with the values of the initial modulus found in other tests.

VARIATIONS IN E WITH RELATIVE CONSISTENCY

Table 2 gave the variation in E of neat cement cylinders as the water-cement ratio varied. Other series of tests in which the materials and proportions of mix were kept constant while W/C varied, were reported by Walker[6]. Fig. 1(a) shows the results from several groups of cylinders made with relative consistencies of 0.80 to 1.50. Since values of secant modulus are not given, initial tangent values are used. The ratio of E/f_c' is plotted against relative consistency. In all cases there is a rapid increase in E/f_c' as the relative consistency increases, showing that added water reduces f_c' much more than it reduces E. Fig. 1(b) shows similar results from another series, for a wide range of mixes. The secant modulus at 0.25 ultimate load is used. The values of E/f_c' range from 365 to 4950. In Fig. 2, values of E from this series of tests have been plotted against f_c'. Points for each mix are connected by lines, the wet consistency being at the left end of each group. Values of the ratio E/f_c' are indicated by radial lines through the origin. The effect of richness of mix will be discussed later.

The effect of consistency upon the E/f_c' ratio was pointed out by Professor Ban[7] in 1929. On the basis of a large number of tests of gravel concrete he derived equations for stress-strain curves and for the value of E, in terms of the compressive strength and the water-cement ratio of the concrete. His results are in general accord with the trends shown in Fig. 1 and 2, the mixes with low water-cement ratios having low values of E/f_c'.

VARIATION IN E DUE TO GRADATION OF AGGREGATE

Several series of 6 by 12-in. cylinder tests have been made in which mix and consistency and character of aggregate were held constant, but the gradation varied. While the water-cement ratio also varied, the consistency of the cement paste in the mixture probably did not vary greatly. Fig. 3 shows data of tests by Walker[6], and a similar series of tests reported in part by Talbot

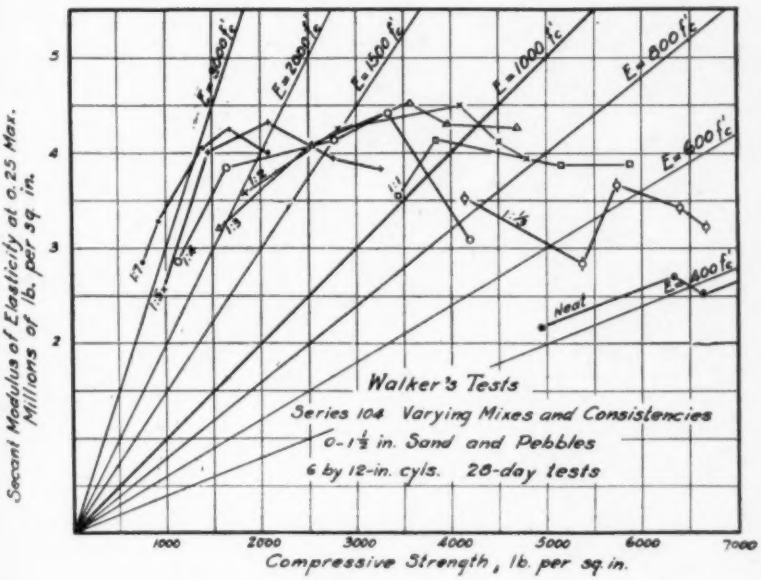

FIG. 2

and Richart[8]. In Walker's tests, mixes of 1.0 or 1.10 relative consistency of 1:3, 1:4, and 1:5 proportions, are used, the aggregates varying from a 0-14 sand to 0-2-in. sand and pebbles. Similarly in the Talbot-Richart tests, a 1:5 mix of normal consistency throughout was used with smoothly graded aggregates varying from 0-100 sand to 0-2-in. sand and gravel.

When variations in strength are produced by changing the gradation of the aggregate, with constant mix, consistency and kind of aggregate, it appears from Fig. 3 that a fairly definite relation (though not a linear one) may be found between f'_c

and E. It is to be noted that the stronger mixes shown although of high density, are rather harsh and unworkable and that there is a tendency toward a constant value of E at stresses of 3000 to 4000 lb. per sq. in. Values of the ratio E/f'_c are seen to be relatively greatest for the mixes of low strength.

VARIATIONS IN E DUE TO KIND OF AGGREGATE

A number of tests have been made in which the effect of different kinds of aggregate was studied. Data of tests by Wig,

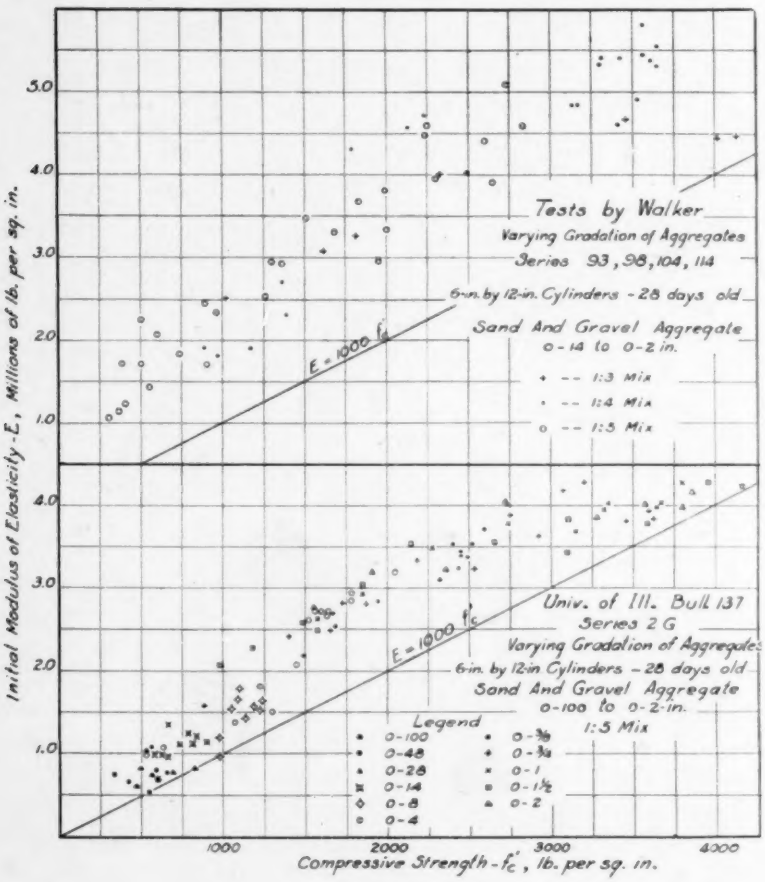

FIG. 3

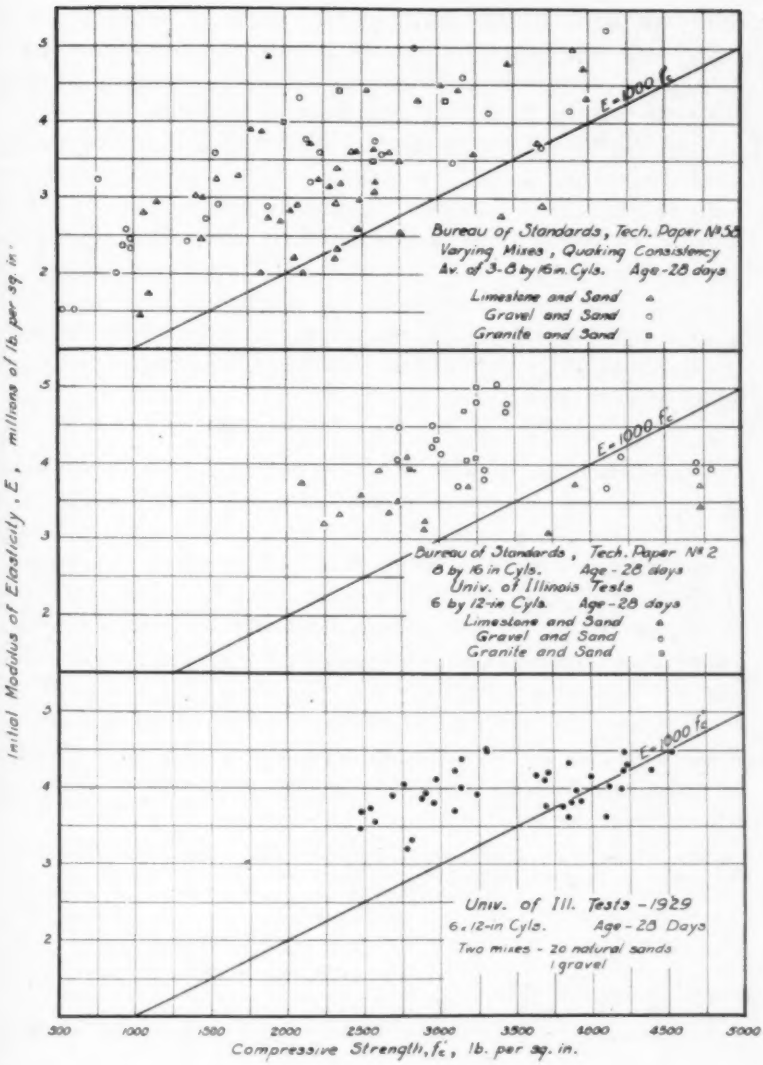

FIG. 4

Williams and Gates[9], Humphrey and Losse[10] and the writers, are given in Fig. 4. In the first two diagrams the coarse aggregate is of various kinds as shown; in the last one, 20 kinds of natural

sands are used with one lot of gravel. Tests by Walker[6] on concrete made with gravel, limestone, granite and slag coarse aggregates show very little difference in either the values of E or of the ratio E/f'_c due to the different kinds of material.

The diagrams of Fig. 4 indicate little effect of any particular kind of aggregate upon the value of E. This is not very consistent with the idea stated earlier in this discussion that E will be affected by the stiffness of the aggregate or else the aggregates used did not have as great a variation in quality as do those in Table 1. The tests in which 20 fine aggregates were used show quite a uniform value of E regardless of the concrete strength. In fact, none of the data of Fig. 4 show much agreement with the relation, $E = 1000\ f'_c$.

VARIATION IN E WITH RICHNESS OF MIX

Among the tests showing the variation of E with the richness of the concrete mixture are those of Davis and Troxell[11] and Walker.[6] Davis and Troxell show values of the secant modulus of elasticity at a stress of 400 lb. per sq. in. of sandstone concrete 40 days old, of 4,000,000 lb. per sq. in. for a 1:3.5 mix, 4,250,000 lb. per sq. in. for a 1:4.5 mix and 3,900,000 lb. per sq. in. for a 1:6 mix. At greater ages the increase in E is greater for the richer mixture, so that at 1000 days the respective values of E for the 3 mixes are 5,350,000; 5,050,000; and 4,700,000 lb. per sq. in. Tests of Series 114, by Walker, give the values of the secant modulus of elasticity at 0.25 ultimate load and compressive strength shown in Table 3.

TABLE 3. MODULUS OF ELASTICITY FOR CONCRETE OF DIFFERENT MIXTURES
Sand and Gravel Aggregates, 0-1½-in. size. Concrete of normal consistency, 28 day tests

Mix, by Vol.	1:9	1:6	1:5	1:4	1:3	1:2
Secant Modulus of Elasticity, E_{25}, in thousands of lb. per sq. in.	3710	3820	4220	4660	4650	4660
Comp. Strength, f'_c, lb. per sq. in.	1070	2060	2680	3310	4030	4870
Ratio E/f'_c	3460	1850	1570	1410	1150	960
Water-cement ratio by vol.	1.23	0.93	0.83	0.73	0.63	0.53

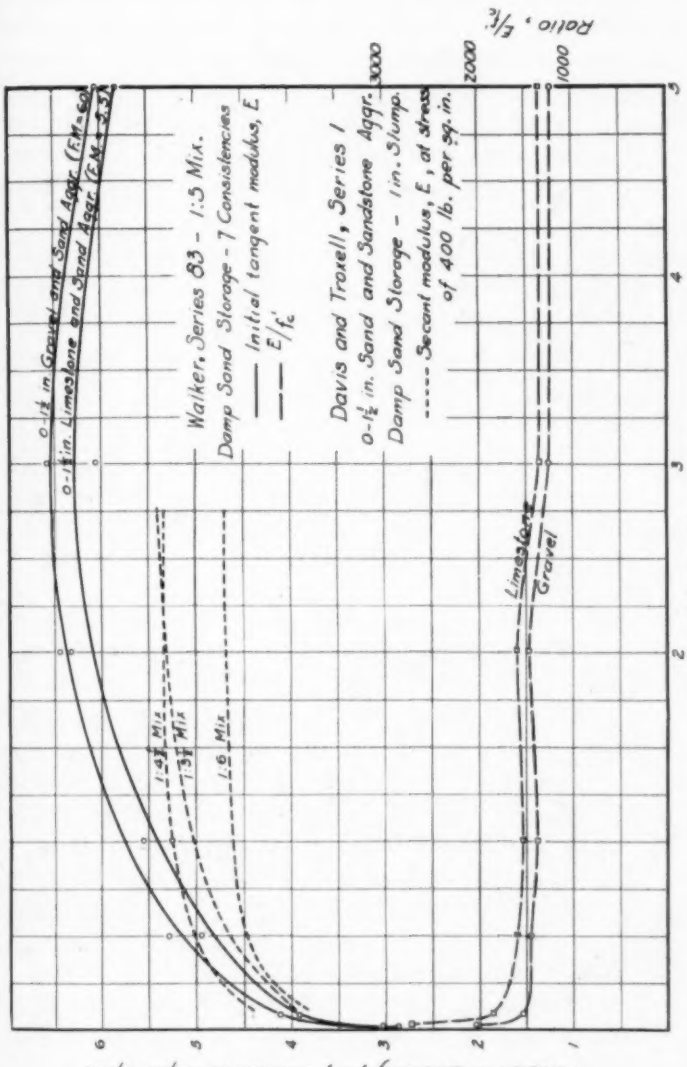
Modulus of Elasticity, E, millions of lb. per sq. in.
Age of concrete, years
Ratio, E/f'c
0-1/4 in Gravel and Sand Aggr. (F.M.=6.0)
0-1 in Limestone and Sand Aggr. (F.M.=5.5)
Walker, Series 83 - 1:5 Mix.
Damp Sand Storage - 7 Consistencies
—— Initial tangent modulus, E
— — E/f'c
1:4.3 Mix
1:5.7 Mix
1:7.6 Mix
Davis and Troxell, Series 1
0-1/2 in. Sand and Sandstone Aggr.
Damp Sand Storage - 1 in. Slump.
----- Secant modulus, E, at stress of 400 lb. per sq. in.
Limestone
Gravel

FIG. 5

The table shows that E is about equal for the 1:4, 1:3 and 1:2 mixes at 28 days. The ratio $^{E}/f_c'$ is seen to decrease as the mix becomes richer, and the strength increases. The effect is similar to that shown in Fig. 2, in which the values of E at normal consistency (second point from the right in each group) increase from the leaner mixes up to a maximum at the 1:2 and 1:3 mixes and then decrease again with the still richer ones. The ratio $^{E}/f_c'$ shows a corresponding decrease from the lean to the rich mixes.

Although the mixes of Table 3 are of normal consistency, the water-cement ratio varies. A rather complex empirical relation between E, f_c', and water-cement ratio has been stated by Ban[7].

VARIATION IN E WITH AGE

While working stresses and other design quantities such as modulus of elasticity are based upon tests of concrete at the age of 28 days, it is of interest to know how E is affected by age and also to know how the action of a structure may be affected by a gradual change in both strength and stiffness of the concrete. The old statement that stress travels the stiffest path applies to a composite member of steel and concrete as well as to an indeterminate frame. The proportionate increase in both f_c' and E generally found with increase in age would appear to be a fortunate provision of nature.

In Fig. 5 is shown the effect of age on initial tangent modulus of elasticity as found by Walker[6] and on the secant modulus of elasticity at a compressive stress of 400 lb. per sq. in., as found by Davis and Troxell[11]. In both cases the tests were made on 6 by 12-in. cylinders, stored in damp sand and tested damp. Walker's tests show a retrogression in E beyond the age of 3 years, while Davis and Troxell find nearly a constant value beyond the age of one year. Walker's tests show a general increase in strength with age, so that the ratio $^{E}/f_c'$, also shown in Fig. 5 is seen to decrease slightly as the age increases beyond 28 days. Tests by Wig, Williams and Gates[9] over a period of one year, and by Johnson[12] over a period of 90 days show a general increase in E similar to those shown in Fig. 5.

It is to be noted that a very high value of E is developed by concrete in moist storage for 1 to 3 years. The continued gain

in strength and density of the cement matrix evidently imparts to the concrete a high value of E that can be obtained in no other way.

MISCELLANEOUS TEST DATA

A number of tests have been reported in which the effect of several variables are hopelessly entangled. For example, a large group of tests were made by Johnson[13] on pavement cores made with a great variety of aggregates, including limestone, trap, gravel, slag, granite, and sandstone of ages ½ to 11 years. The upper diagram in Fig. 6 shows values of E from these tests plotted against the compressive strength, f'_c. As might be expected, there is no definite relation between the two to be found from the data. A second group of data are taken from tests by Withey[14], in which the age of cylinders varied from 35 days to 65 days and the mix varied from a 1:3:6 sand and limestone concrete to a 1:1.29 sand mortar. It is notable that values of E were nearly constant for concretes for which the strengths varied from about 3000 to 8000 lb. per sq. in.

APPLICATIONS OF E IN FLEXURAL DESIGN

Little has been said in the preceding discussion regarding the gradual decrease in the secant modulus of elasticity with increase in stress, but there is a wealth of data on this subject. In view of this decreasing value, some writers have advocated the use of a value of E for design much less than the initial tangent modulus found by test. Perhaps as good a treatment of the variation in E as any is that embodied in the parabolic theory of flexure of reinforced concrete beams developed by Talbot[15]. According to this theory the initial tangent modulus is employed, and the decrease in E with increase of concrete stress is taken care of by the assumption of a parabolic stress-strain relation. If working stresses in flexure are chosen with a full consideration of the parabolic stress relation existing at the ultimate load on the member, a further compensation by the use of a low value of E becomes unnecessary.

Regarding the value of E to be used in determining the relative stiffness of rigidly connected columns and girders in frame analysis, it should be remembered that in general only relative values

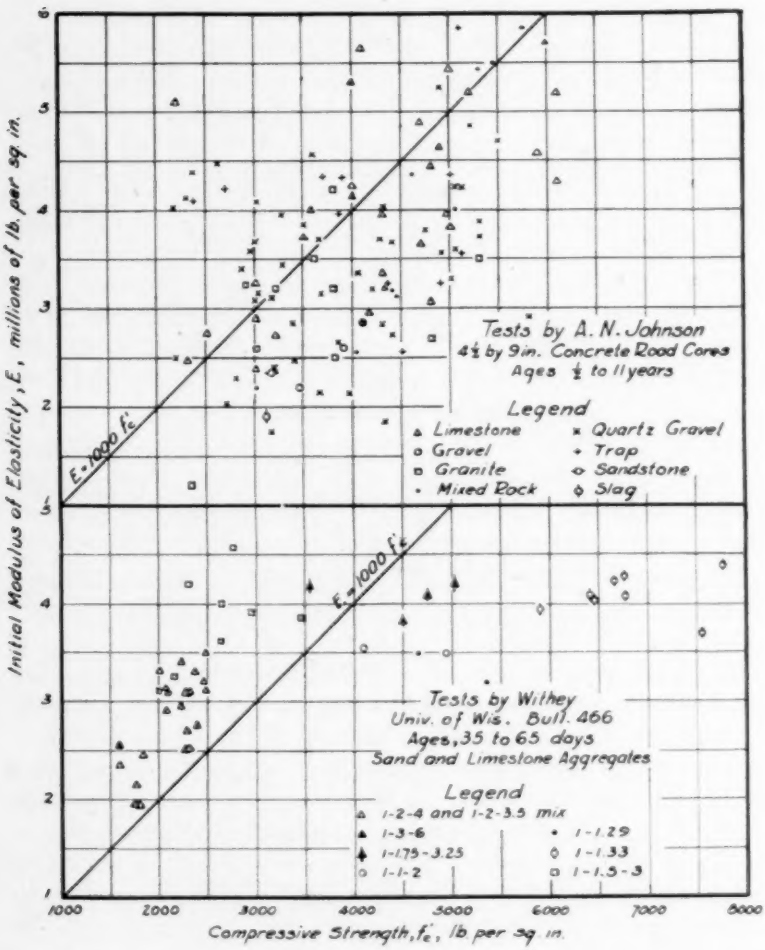

FIG. 6

of E for the different members are needed and it seems logical to use for this purpose the initial test values.

Column formulas in common use involve the quantity, n, or ratio of E for steel and concrete. For a study of stress distribution at working loads, which is not generally of particular importance, such a formula may be useful, but for ultimate loads

it is doubtful if the ratio, n, has any particular significance. The factor of safety of a column depends upon the ultimate load that can be carried, and it is quite well established that the ultimate load is a function of the strength of the concrete and of the stress near the yield point of the reinforcing steel. At the ultimate load on a spirally and longitudinally reinforced column, there is so much plastic deformation that the ratio of elastic moduli, n, has no meaning.

CONCLUSION

This discussion was intended to show that while the effect of certain variables upon the modulus of elasticity E, may be isolated and studied, it is hardly possible to express the value of E as a constant times the compressive strength, f'_c.

In general, values of $^{E}/f'_c$ are relatively high for the weaker concretes, as produced by wet consistencies, lean mixes, fine gradings of aggregate and early ages. High absolute values of E are obtained by long curing, by the use of well-graded aggregates of selected quality, by the use of fairly stiff consistencies and medium richness of mix.

For a concrete for which only the strength, f'_c, is specified, the relation $E = 1000\ f'_c$ may be greatly in error. From the data of this discussion it appears that the value of E for 28-day concrete of the type of mixtures in general use in construction will not vary greatly for a considerable range of strengths. Values of E varying from 3,000,000 lb. per sq. in. for 2000-lb. concrete to 4,000,000 lb. per sq. in. for 3000-lb. concrete and stronger represent the range to be expected. The corresponding values of n range from 10 to 7.5.

The comparatively small range in n just stated leads to the thought that a constant value of n, such as 9, might be used with less error than that existing under the present code provisions. In some ways a constant value of n would seem advantageous. The use of high strength concrete in exterior building columns and in rigid frames would be quite effective since increasing the strength of the concrete would not increase the stiffness and thus would not increase the induced bending moments which are so often difficult to handle. For the case in which $\frac{EI}{l}$ for columns is much less than that of the adjoining girders, a large bending

moment could thus be provided for by increasing the column strength (without increasing the moment to be carried) or better still by increasing the concrete strength and decreasing the column area, thus decreasing to a greater degree the value of I and of the corresponding induced bending moment.

The use of a constant value of n would introduce some simplification of beam design, though the effect of a considerable variation in n may be shown to have comparatively little effect upon the actual design of a beam section.

Whether or not the value of n should be considered constant, it seems certain that it should not vary as widely as is indicated by the equation $E = 1000\ f'_c$. Hence, in several parts of the paper by Prof. Cross where, in accordance with the Joint Building Code, E has been considered proportional to f'_c, some modification of the argument may be advisable. Thus the variation of $\frac{EI}{h}$ is not necessarily proportional to values of $\frac{f'_c I}{h}$, as stated in the introduction. Neither is it certain that an increase in concrete strength in eccentrically loaded columns will actually weaken the column as indicated in the section on high strength concrete.

The suggestion is made by Professor Cross that the resistance of concrete to bending is limited, not by its strength but by the deformation produced; assuming the column to be subject to a practically irresistible rotation. The statement is frequently made that all concretes fail in compression at a unit deformation of about 0.0015. However, there are some known exceptions to this generalization. Data from neat cement cylinders given in Table 2 show unit shortening at maximum load as great as 0.0036. These cylinders were very "rubbery" and would recover a large part of this deformation if the load were removed before the cylinder actually broke. They also broke violently at failure due to the large amount of energy stored in them. While these neat cement cylinders do not represent concrete in practical use, they are cited as an indication that very rich concretes will have a greater ultimate deformation than the oft-quoted value of 0.0015. Another exception to the rule is found in beam tests in which the compressive strength of the concrete is actually developed. Slater and Zipprodt[16] gave data on the ultimate concrete deformation of 23 beams having high compressive stresses;

of these 8 showed unit deformations of 0.0025 to 0.0033 and the remainder from 0.0017 to 0.0024. Talbot[15] showed the results of several highly reinforced concrete beams in which the concrete deformation at ultimate load was as much as 0.0020 to 0.0030. It seems logical that when the deformation of an extreme fiber is involved the adjacent fibers offer enough support to permit a large amount of plastic deformation at the surface, as explained by the second and fourth degree parabolic variation of stress used respectively by Talbot and Slater. It appears, then that the assumption of a constant maximum deformation in eccentrically loaded columns of different mixes and of different proportions of direct and bending stress may not be a reliable criterion of ultimate strength.

This paper by Professor Cross has raised many questions of interest to the forward-looking designer, and has presented some new viewpoints that are sure to be very helpful. It is hoped that the discussion will bring out further information on physical properties of the materials of construction which will supplement the logical structural analysis presented.

LIST OF REFERENCES

1. "Final Report of the Joint Committee on Concrete and Reinforced Concrete," 1916.
2. "Standard Specifications for Concrete and Reinforced Concrete," 1924.
3. "Joint Code, Building Regulations for Reinforced Concrete," Proc. Am. Concrete Inst., 1928.
4. "Tests of Metals," Reports 1890, 1894, 1895.
5. Unpublished tests, by F. E. Richart, University of Illinois, 1919.
6. "The Modulus of Elasticity of Concrete," by Stanton Walker, Proc. A. S. T. M., Vol. XIX, Part II, 1919.
7. "Elastic Properties of Concrete under Low Compressive Stress," by S. Ban, Journal, Institute of Japanese Architects, Vol. 43, No. 524, 525, 526, 1929.
8. "The Strength of Concrete, Its Relation to the Cement, Aggregates, and Water," by A. N. Talbot and F. E. Richart, Bul. 137, University of Illinois, 1923.
9. "Strength and Other Properties of Concrete as Affected by Materials and Methods of Preparation," by R. J. Wig, G. M. Williams and E. R. Gates, Tech. Paper No. 58, U. S. Bureau of Standards, 1916.
10. "Strength of Reinforced Concrete Beams," by R. L. Humphrey and L. J. Losse. Tech. Paper No. 2, U. S. Bureau of Standards, 1911.
11. "Modulus of Elasticity and Poisson's Ratio for Concrete, and the Influence of Age and other Factors upon these Values," by R. E. Davis and G. E. Troxell, Proc. A. S. T. M., Vol. 29, Part II, 1929.
12. "Relationship Between Strength and Elasticity of Concrete in Tension and in Compression," by J. W. Johnson, Bul. 90, Iowa State College, 1928.
13. "The Modulus of Elasticity of Cores from Concrete Roads," by A. N. Johnson. Public Roads, Vol. 9, No. 8, October, 1928.

14. "Tests of Reinforced Concrete Columns," by M. O. Withey, Bul. 466, University of Wisconsin, 1910.
15. "Tests of Reinforced Concrete Beams," by A. N. Talbot, Bulletins 1 and 4, Eng. Expt. Sta., University of Illinois, 1904, 1906.
16. "Compressive Strength of Concrete in Flexure," by W. A. Slater, and R. R. Zipprodt, Proc. A. C. I., Vol. XVI, 1920.

AUTHOR'S CLOSURE

BY HARDY CROSS

The two papers which the writer presented in the December *Proceedings* are closely related to each other. The exact analysis of moments in rigid frames depends a good deal on the assumed action of the columns in flexure. On the other hand, it is evidently futile to compute the bending moments in the columns with more precision than can be justified in the design of these columns. The discussions of the two papers have served to bring out very clearly several important aspects of the subject.

Much of the fascination of the study of reinforced concrete design arises from the fact that problems must be considered from several angles in order to develop clear ideas for design. Neither laboratory experiment, nor field observations nor mathematical analysis nor detailing against all possible modes of failure can be called the only scientific method of study, nor has anyone of them shown uniformly superior results as compared with the others. Each of them has at times proved powerfully illuminating and at other times very misleading . Nowhere is this more true than in the study of concrete frames.

It is beginning to be generally recognized that the exact details of the design of beams and columns for known moments and forces has been carried to great extreme compared with the problem of what moments and forces these members have to resist. On the other hand, there is a tendency in the hands of many students to carry the elastic analysis to a point which is futile in view of uncertainties as to the action to the structure.

The paper on Flexure of Columns points out certain consequences of the existing Building Code of the Institute. It was intended to indicate questions which may form a basis for further discussion. Both papers were written largely from the viewpoint of one who is interested primarily in the design of structures and addressed to men who are concerned, as Mr. Cutler is, with

definite problems affecting construction; but they involve also several matters of great interest to research men.

Mr. Roy and Professor Richart have reported most valuable studies. Apparently the modulus of elasticity of concrete is erratic, but more or less constant from one job to another so far as the designer is concerned. Two slightly different questions are involved: the first is the prediction of the modulus of elasticity of concrete for a given strength at a particular job; the second is the question of how this modulus will porbably vary with variation in the strength of the concrete at this job. It has always seemed to the writer most improbable that an engineer can predict with assurance that in some structure which will be built from his design next month in another section of the country, the modulus of elasticity of concrete will have a definite value of three million pounds per square inch. This does not, however, affect the question of whether this modulus, whatever its value, will probably be increased or decreased by increasing or decreasing the strength of the concrete at this job.

There is a good deal of evidence indicating that increase in strength under given conditions will usually be accompanied by increase in modulus. Some of the data presented by Mr. Roy and Professor Richart indicate that this increase will not continue indefinitely, but only for moderate increases in strength. But it appears from the data which they have presented that quantitative statement of the relation of increase in modulus to increase in strength is not now possible. Certainly, there is enough difference in the evidence from different sources to dictate caution in the acceptance for general application of the values referred to by Professor Dunagan.

The paper indicates that the assumption as to the relative modulus of girders and columns affects very largely the resulting moments in the columns. On the other hand, it should be emphasized that it does not ordinarily affect very much the resulting moments in the girders. This is indicated in the report in the statement that whereas the moments in members transversely loaded are usually affected very little by uncertainties as to the relative stiffness of the different members, the moments in members not loaded transversely, such as columns, are very much affected by the relative stiffness.

Mr. Cutler's valuable contribution brings to the front another point in the design of reinforced-concrete columns. The axially loaded column of reinforced concrete is the exception and not the rule. Our codes have been drawn to give in great detail methods of design for axial loaded columns and have added supplementary comments dealing with flexure. At the present time we cannot perhaps do much better than this. Large numbers of building columns, however, are subject to temperature stresses of several hundred pounds per square inch and usually to additional flexural stresses of another two or three hundred pounds per square inch from connecting members, so that columns which are designed as axially loaded actually have four hundred or five hundred pounds bending stress in them. It is true that the writer personally knows of no specific evidence of difficulty from this source in existing structures, although he has seen the statement made several times that secondary flexure has caused serious cracking of certain building columns. It is difficult, however, to definitely identify a crack in a building column as being the result of overstress.

Mr. Roy and Professor Richart indicate some doubt as to the probability that increased moment will be accompanied by proportional weakening of the column. They refer to the parabolic theory of stress distribution as indicating their viewpoint. Has our code already discounted the yielding of the outer fibres by the increased stress permitted in the column for combined axial load and flexure? That stress from flexure is less dangerous than from axial load seems logical but we have few test data. If the figures shown by Mr. Dell are replotted so that the computed values pass through the axial load which produced failure, they will be found to coincide nearly exactly with the curve of loads at failure for different eccentricities. This does not indicate any substantial relief of the outer fibres of the concrete when the column is flexed. These tests indicate very well that the moment is quite as important as is the axial load.

The writer has heard the tests reported by Mr. Dell cited in support of the theory of the cracked section and of the cubic equations exhibited in our literature. The proximity of the two curves for design drawn by Mr. Dell shows why these data can not be used in support of either the theory of the cracked section

or of the uncracked section. Mr. Dell's contribution confirms the writer in his opinion that reinforced concrete columns in flexure can be designed quite well enough by the formulas given in the paper.

Another phase of the whole problem of continuity in concrete frames needs further consideration. The flexural stresses in the columns are of the nature of what is called secondary in steel design. This means that they are objectionable but are not usually depended on to carry the load, and that consequently the beginning of failure from this source will not at once involve failure of the structure because the load will not so definitely follow up such initial failure. This seems an important distinction between secondary and primary stresses. We are justified in using higher stresses where the stresses are not counted on to carry the load. When we depend upon such stresses to actually support the structure, we should correspondingly reduce the allowed stress to that permitted for primary stresses. Under existing specifications it is sometimes more economical to depend upon flexure of the columns, using the increased permissible stresses in columns in flexure, than it is to carry the load by flexure of the girders. If this is done, the column flexure is primary.

The writer hopes that these papers may be of some service to those who are really interested in correct solutions of the problems involved. But the whole subject will not be cleared up in a day, for several of the problems have apparently been studied little and there is no general agreement on most of them. What is needed now is further discussion and study—not immediate standardization.

With this view Professor Hickerson probably does not agree. Nevertheless the writer is convinced that too hasty standardization in this field will be unfortunate. The preparation of office standards for the case of equal span lengths need not be very laborious or expensive, but such standards will be rather complicated if they recognize the important variables, (a) relative stiffness of columns, (b) relative width of columns, (c) relative intensity of live and dead load. If they do not recognize these, they will create the same false feeling of accuracy as do the elaborate curves for the analysis of rectangular columns in flexure when they neglect the existence of side steel in the column.

The better class of engineers will prefer to be quite clear as to the basis of computation of such standards and will check them before using them. In order to be useful, then, such standards must state very clearly:

(1) For what conditions of loading the maxima are computed. It seems to be generally agreed to neglect the effect on the girders of loads on other floors, but there is no agreement on this point with regard to column moments. There is also some difference in practice with regard to combinations of loads on one floor for maximum girder moments.

(2) What assumption is made as to flexural action along the girder. Is the girder treated as a prismatic member between centers of column axes or is it assumed that no deformation takes place in the girder beyond the face of the column or is some compromise adopted? This is an important matter as discussed later. Any set of standards which gives moments based on the clear span but neglects the ratio of total span to clear span is entirely misleading and probably worthless.

(3) Whether L is to be taken for the clear span or for the total span.

The writer looks forward with interest to the publication of Professor Hickerson's curves. But whatever specifications he lays down for their preparation, he will be fortunate indeed if he does not find many engineers who disagree with him on some detail. What the writer hopes for is a permissive clause encouraging the use of elastic analyses. Some of them will not be quite correct, but the designs will probably be no worse than those made with conventional coefficients. After differences of opinion are straightened out, more definite specifications may be drawn. At the present time most designers think that an exact analysis of a rigid frame is a purely mathematical procedure involving no assumptions whatever. It should be clearly understood that where the columns have appreciable width, analyses made by slope-deflection, for example, are not exact.

The writer wishes, then, to emphasize further one point in the Report on Rigid Frames. On page 175 there is a paragraph dealing with "Span Length." Ordinary methods of analyzing rigid frames, such as slope-deflection, take no cognizance of the

ratio of clear span between centers of columns. Many analyses of rigid frames have been presented as exact analyses in which the moments are computed on the clear span and the structure is treated as if it were a line diagram having these clear spans. This is definitely wrong and may lead to quite large errors in design moments. To attempt to exactly analyze a rigid frame on any such basis is a waste of time and money, because it represents no conceivable action of the structure.

No attempt was made in the report to allow for the ratio of total span to clear span, the girders being treated as of uniform section to the center line of the column. Correction for the effect of the merging of girder and column may be made approximately by increasing the fixed-end moment in the ratio of total span to clear span. The fixed-end moment for uniform load is $1/12\ wL^2$, where w is the load per foot and L the span center to center of columns, if the columns have no width. But if the columns have a width equal to 1/10 of the span, this fixed-end moment may be taken as approximately $1/11\ wL^2$, and for a width of column equal to 2/10 of the span, the fixed-end moment may be taken approximately as $1/10\ wL^2$.

All analyses—by any method—must be based on the total span. It does not make very much difference what the exact values of the relative stiffnesses are nor what the carry-over factor is, but it makes a great deal of difference what the fixed-end moment is. It may be added that, if the columns have appreciable width. the stiffness is *not exactly* proportional to $\frac{I}{L}$ *nor* is the carry-over factor *exactly* equal to $-\frac{1}{2}$.

The variable ratio of total span to clear span is much more important in determining moments in the girders than are small differences in the relative values of I. If Mr. McCormick will redesign his frame assuming no flexural deformation in the girder after it joins the column face or in the column above the bottom of the girder—or on any reasonable assumption of this nature—he will discover other elements in the problem besides sidesway. Mr. McCormick, however, has correctly interpreted the statement twice made by the writer in the report, that the Method of Moment Distribution, as it is given there, neglects sidesway. In

the writer's paper "Continuity as a Factor in Reinforced Concrete Design" he will find sidesway discussed and a method of allowing for it is there presented with an example. But Mr. McCormick is entirely wrong in thinking sidesway an important factor in design for vertical loads, even in the peculiar case which he shows and on the assumption that his bent is free to sway. Ordinarily such a bent would be connected to other bents by a slab which would resist strongly sidesway of any one frame.

Finally, attention is called to the generally profound significance of the contribution by Mr. Roy and Professor Richart. Do we really know anything consistently about the modulus of concrete? Are our nice computations for amount of compressive reinforcement in girders and in column wasted? Does it really make any great difference? It does not materially affect the amount of tensile steel. If it is correct that in compression the ultimate strength of the concrete is added to the yield point strength of the steel before failure, the value of n is of no great importance in reinforced concrete design and we may as well discard much of the elaborate algebra which occupies so much space in the text-books. The present tendency is distinctly in that direction. A simpler and less fussy theory is being evolved, but one which is a more useful tool to the designer.

ABSTRACTS

of the current literature of cement and concrete

PREPARED UNDER THE DIRECTION OF THE
AMERICAN CONCRETE INSTITUTE
COMMITTEE ON ABSTRACTS

J. C. WITT, Chairman, (*Materials*)
H. F. GONNERMAN (*Properties of Concrete*)
F. E. RICHART, (*Engineering Design*)
REXFORD NEWCOMB (*Architectural Design*)
THEODORE CRANE (*Field Construction*)
W. D. M. ALLAN (*Shop Manufacture*)

EDITED BY DWIGHT F. JENNINGS

ABSTRACTORS THIS MONTH

JOHN E. ADAMS, Ealing, England.

F. O. ANDEREGG, Senior Industrial Fellow, Mellon Institute of Industrial Research, Pittsburgh, Pa.

C. BACHMANN, Editorial Dept., *Concrete*, Chicago, Ill.

A. E. BEITLICH, Whiting, Ind.

JAY BOHN, Research Dept. *Rock Products*, Chicago, Ill.

A. A. BRIELMAIER, University of Illinois, Urbana, Ill.

H. FRAUENFELDER, Algonite Stone Mfg. Co., St. Louis, Mo.

A. J. HOSKIN, Editor, *Pit and Quarry*, Chicago, Ill.

F. H. JACKSON, U. S. Bureau of Public Roads, Washington, D. C.

V. P. JENSEN, University of Illinois, Urbana, Ill.

D. E. LARSON, University of Illinois, Urbana, Ill.

N. H. ROY, University of Illinois, Urbana, Ill.

G. M. Williams, Professor of Civil Engineering, University of Saskatchewan, Saskatoon, Saskatchewan.

MATERIALS

ADMIXTURES

The effect of trass on the setting time of cements. A. STEOPOE. *Tonind. Ztg.* (Germany), Feb. 20, 1930. V. 54, No. 15, p. 237-9.—Trass from Slanic and Dej, Rumania, when added to standard portland cement, to high early strength cement and to Kühl cement increased water requirement in all cases and slowed up setting reactions, except with last, with which an acceleration was noted.—F. O. ANDEREGG

Porcelain aggregate as an admixture for mortar. HEINRICH LUFTSCHITZ. *Tonind. Ztg.* (Germany), Feb. 24, 1930. V. 54, No. 16, p. 255-7.—Crushed porcelain may be added to mortar to give an especially fine product, strong and resistant to abrasion.—F. O. ANDEREGG

Rhenish trass as a hydraulic admixture for concrete and mortar. *Das Betonwerk,* (Germany), Feb. 9, 1930, V. 18, No. 6, p. 73. With the exception of lime no binder has an older history than trass. The durability of trass is unquestioned; for example, the Roman construction in the Rhine Valley. The outstanding quality of a trass lime mortar consists of its high elasticity and resistance to water action. The use of trass in cement concrete is recommended because it increases plasticity and resistance to disintegration. It increases hardness and combines with free lime transforming it into silicate of lime. The admixture of trass prevents segregation. Hair cracks are prevented by its use and premature drying out prevented. It is also claimed that the hardening temperature is lowered.—H. FRAUENFELDER

Admixtures to waterproof concrete. *Das Betonwerk* Jan. 5, 1930, V. 18, No. 1, p. 3.—Formulae for several kinds of admixtures are given, including, first, aluminum steareate. Use of this material integrally fills pores with water repellent substance without hindering setting of cement. Second, a watery paste of 40 per cent barium carbonate, 15 per cent oleic acid, 10 per cent sodium silicate (thinned with ½ its volume of water) and 35 per cent water. This paste should not only make concrete watertight but also prevent efflorescence. Third, a mixture of fine sand, common laundry soap and alum. Soap should be composed of 50 per cent soap, 40 per cent water and 10 per cent sodium silicate. One part soap dissolved in boiling water is added to 3 parts fine sand by weight. This mixture is allowed to cool and become hard, then ground to a fine powder. To each 2 parts of this powder, 1 part powdered alum is added. Resultant powder is used in the proportion of 2½ kilograms to 1 cubic meter of concrete, filling pores with water repellent aluminum oleate. Fourth, a mixture of sodium carbonate and aluminum sulphate, proportioned 1 to 1½ is added to 100 parts cement. Treated cement is used in regular way. Fifth, other very complicated formulae are described, which make use of china clay, sodium silicate and calcium chloride.—H. FRAUENFELDER

Cement admixtures — a comment. ROY CROSS. *Rock Products,* Feb. 15, 1930, V. 33, No. 4, p. 75-76.—In all masonry work, hydrated lime or some other admixture has always been demanded by the actual user of cement. Admixture controversy is seemingly rejuvenated with each new generation. Much of misunderstanding has been caused by lack of satisfactory laboratory method of determining workability of concrete. Conclusions drawn by Prof. Abrams in recent report ignore water-cement ratio. From data taken from Abrams' paper in Proc. of Am. Soc. for Testing Materials, V. 29, part 2, 1929, arranged with same water-cement ratio, and admixture designated in Prof. Abrams' recent conclusions to be most injurious, results show in practically all instances, action of bentonite increased compressive strength. Slump test is not a measure of workability. Literature relative to effect of puzzolanic materials on portland cement, seemingly has been overlooked. Paper

presented by Andreas Lundteigen, April 7, 1897, (c.f. Notes on Portland Cement and Concrete, *Trans. Am. Soc. C. E.*, No. 807) sets forth conditions under which admixture of trass improved portland cements for certain purposes. Effect of volcanic silica derived from volcanic ash used by Mr. Lundteigen is to react with excess of lime developing during hardening and setting of cement. One table is included. —JAY BOHN

Sodium silicate as an addition to cement. H. NITZSCHE. *Zement* (Germany), March, 1930, V. 19, No. 10, p. 213-4. — Experiments concerning the influence of sodium silicate on the resistance of portland cement and blast furnace slag cement against sulphate solutions. The cements were mixed with sodium silicate up to 6.5 per cent and testpieces were made with standard sand (1:4). The changes in the length of the testpieces, which were stored in water and a 5 per cent $MgSO_4$—solution, were observed. Portland cement showed improvements with additions of 1.5 and 6.5 per cent. The samples of blast furnace slag cement showed deterioration.—A. E. BEITLICH

The effect of calcium chloride on concrete. WALTER OBST. *Zement* (Germany), Feb., 1930, V. 19, No. 9, p. 203-4.—Additions of $CaCl_2$ to concrete prevent freezing of the latter during cold weather conditions. The strength properties are much influenced, especially with a higher percentage of $CaCl_2$. The initial strength is increased with increasing amount of $CaCl_2$ but retrogression takes place in later ages. The author claims 2 per cent as the best admixture and advises not to exceed 3 per cent.—A. E. BEITLICH

AGGREGATES

New quarry and plant near Orofino, Idaho, prepares crushed and pulverized limestone. *Pit and Quarry*, March 12, 1930, V. 19, No. 12, p. 67-70.—Plant produces chiefly limestone products, but plans are laid for the manufacture of lime. Article describes geology, quarrying, transportation of rock by an aerial tramway and the crushing, screening and pulverizing operations of the Clearwater Lime Products Co. Thirteen illustrations.—A. J. HOSKIN

Contraflow gravel washer. *Engineer*, Apr. 11, 1930, V. 149, No. 3874, p. 413.—Gravel washing plant installed in Buckinghamshire, has capacity of 30 tons per hr. Washing is by the cascade system. Drum washer, 5 ft. 6 in. diameter, supported on rollers in phospher bronze bearings, is rotated at 4 revs. per min. by chain drive. Gravel is propelled forward by continuous helical blade on inner side of drum, width of blade varying from 6 in. at charging end to 1 ft. at outlet. Water is introduced at opposite end and falls in series of cascades in face of oncoming gravel, towards delivery end, where it discharges over a lip. Only clean water meets clean gravel. At discharge end, gravel is lifted by radial paddles and dropped over concentric conical chute, which leads it into screens. First part of screens is solid cylindrical plate, with means of discharging whole of gravel unscreened if required. Beyond this are 3-16th-in. and 1½-in. screens, separation being helped by separate water jet. Sand drops into hopper fitted with partition plate, where it falls to the bottom on one side of the baffle plate. Small bucket elevator removes settled sand from other side of partition which is to prevent undue agitation of water in hopper. Stones are prevented from jamming in perforations in screens by roller mounted on separate spindle, which presses inwards against any pebbles which project through the holes.—JOHN E. ADAMS

Gravel washing plant. *Cont. Jour. Specif. Record* (England), Apr. 9, 1930. V. 102, No. 2652.—New gravel washing crushing and grading plant has been opened near Swindon, for working extensive deposit of limestone gravel. The power unit consists of 30 B. H. P. crude oil engine, vertical two-stroke type. Water supply consists of gravity flow from tank fed by pump delivering 3000 gal. per hr. Water from washer is discharged into an open culvert which is rectangular in cross section and contains several right angle bends. The

cross section and bends have been determined by experiment to give just that velocity to escaping water which will deposit sand in suspension but not the finer particles of clay. Clean sand is collected at intervals and muddy water flows into a settling pit. The engine room contains centrifugal water pump, hand operated air compressor and two bottles, for starting.—JOHN E. ADAMS

Sea salt as cause of decay of limestone. P. W. BARNETT. *Arch. Jour.* (England), Apr. 9, 1930, V. 71, No. 1838, p. 586.—Although there is evidence that sea salt is carried inland by wind accidental direct contamination with sea water, either in the quarry or truck, is more probably the cause of decay in Portland stone. Stone firms are therefore advised to see that no stone which has been in direct contact with sea water is put on market.—JOHN E. ADAMS

Need for simplification of sizes in sand and gravel industry. F. H. JACKSON, *Rock Products*, Feb. 1, 1930, V. 33, No. 3, p. 81-83.—Efforts have been largely misdirected to present time endeavoring to establish single series of standard material sizes. There appears to be no real engineering reason for this. Efforts of producers, consumers, public officials and professional engineers should be used to promulgate set of standards for each large producing area, taking into consideration natural characteristics of materials and writing specifications so that best application of these characteristics could be made. Engineer should be able to design mixture of required quality regardless of exact grading of aggregate furnished. He should have assurance that same size and grading upon which he established his design, can and will be furnished to job from start to finish. No need for national standardization. Primary gravel sizes recommended are: No. 4 to ½-in.; ½-in. to ¾-in.; ¾-in. to 1-in.; 1-in. to 1½-in.; 1½-in. to 2-in. Specifications of various users for any aggregate for given purpose must be same in any given production region before problem is really solved.—JAY BOHN

CEMENT

New quarry and improvements have modernized west's pioneer wet process cement plant. FRANCIS CHURCH LINCOLN. *Pit and Quarry*, Mar. 12, 1930, V. 19, No. 12, p. 41-48.—Gives history of Olympic Portland Cement Company's plant at Bellingham, Wash., since erection in 1912, with its many improvements. Limestone quarry face is 75 ft. high and 500 ft. long, worked in three benches. Aerial tramway handles crushed rock from surge bin down to railroad, a vertical drop of 1,000 ft. and horizontal distance of 1,800 ft., where hopper-bottom railroad cars transport it to cement plant. Clay is also shipped in by rail. Raw material grinding is done in three stages including tube mill and ball mills. Slurry is fed to kilns by slow-motion plunger pumps. Air-cooled clinker passes to storage, thence to bins above the finish mill. Cement is withdrawn from silos to screw conveyor in tunnel and elevated to steel bins over packers. Filled bags are belt-conveyed to railway cars. The company is British. Fourteen illustrations.—A. J. HOSKIN.

Oklahoma Portland Cement Co. conducts systematic safety practices. H. W. MILLER. *Pit and Quarry*, March 12, 1930, V. 19, No. 12, p. 84. This plant's safety director discusses history of the safety activities which began in 1925 and the methods of interesting employees.—A. J. HOSKIN

Marquette Cement Mfg. Co. speeds up service by new distributing system. *Pit and Quarry*, March 26, 1930, V. 19, No. 13, p. 19-22.—Self unloading barges used systematically for transport of bulk cement from plant at Cape Girardeau, Mo., to plants in St. Louis, Mo. and Memphis, Tenn. At Cape Girardeau plant, loose cement is handled by pneumatic pump through large rubber hose to specially-designed hopper-shaped railroad cars which are hauled to dock. Here electrical connection is made and longitudinal screw conveyor at bottom of each car feeds cement to another pumping system which conveys it through pipe line to barges. Each of four new all-steel barges has capacity of 800

tons, and is equipped with collision bulk-heads at each end. One barge is loaded at plant while two are being unloaded at terminal plants and fourth is enroute. Unloading is by means of 9-in. screw conveyors at bases of the hoppers, a 12-in. cross conveyor and pneumatic pump which forces cement through pipe line and flexible hose to shore-line pipes leading to concrete storage silos above packing room. Memphis silos have capacity of 15,000 bbl. and St. Louis silos, 30,000 bbl. Nine illustrations.—A. J. HOSKIN

New Orleans plant of International Cement Corp. remains one of leading plants in South. *Pit and Quarry*, March 26, 1930, V. 19, No. 13, p. 37-47.—All raw materials and finished product are shipped by water. Quarry is at St. Stephens, Ala., about 275 mi. by water from the plant. Lime rock and shale are excavated, without drilling and blasting, by electric crawler shovels, and hauled to crusher house where the rock passes through a hammer-mill and onto a belt conveyor which carries it to barges. Three barges, with combined loads of 3,600 tons, are towed by diesel towboat and tug. Silica sand dredged from nearby lakes, is hauled to the plant on barges. Pug mill reduces shale and limestone. Complete mixture of rock and sand is ground by Kominuters, discharged to bucket elevator, to a mixing trough, to tube-mills. Ground material is elevated to three concrete slurry correcting basins, thence to blending basins. Slurry feeds by gravity to elevators, to trough with horizontal agitator along bottom. Ferris-wheel feeds slurry to pot where it is mixed with dust recovered from kiln chamber. Kilns are fed by gravity and discharge to rotary coolers. Clinker passes to preliminary tube mills, thence to finish tube-mills. Screw conveyors distribute cement to storage silos. Conveyors move sacked cement to railroad cars or trucks. Thirty-one illustrations.—A. J. HOSKIN

Internationalization of the portland cement standards—Part II. C. R. PLATZMANN, *Rock Products*, Feb. 1, 1930, V. 33, No. 3, p. 60-61.—There is evidence of keen interest in all parts of world in international standardization of portland cements by producers and consumers. There is need of emancipation from multitude of standards set up by different countries. Much of respective specifications could be retained. Probability exists of increasing minimum value of hydraulic modulus from 1.7 to 1.8-1.9. Maximum admissible magnesia content of 5 per cent, and maximum content of 2.5 per cent sulphuric anhydride are to be permitted. Specifications concerning specific gravity, ignition loss and insoluble residue can be considered as unnecessary. Figure of 3 per cent for admixtures for special purposes should be retained. Conformity now exists in Vicat needle apparatus, height of specimens and needle weight. There is far less harmony regarding setting, as such, and manner of determining start of setting. Complications arise in testing for soundness. Degree of fineness of grinding must receive consideration. Tests for compressive and tensile strength will have to be binding. Preparation of test specimens will have to be standardized. Customary specifications governing amounts of mixing water may be retained. Initial movement towards international standardization considers only portland cement. Slag cement, blast furnace cement, high early strength cement, and alumina cement to be considered later.—JAY BOHN

Delivery under "more or less" contract for sale of cement. LESLIE CHILDS. *Rock Products*, Feb. 1, 1930, V. 33, No. 3, p. 61.—Differences of opinion and considerable litigation have followed on terminology of contract for sale of cement to be used upon particular job, and quantity to be delivered is named, followed by the phrase "more or less." This subject of law of sales cannot be covered by any hard and fast rule, because each case of this kind has been decided upon facts involved. It is a question of whether the quantity named, or buyers' requirements for particular job will govern. Generally speaking where purchase is made for particular purpose, and buyer is obligated to use only material from seller, a "more or less" provision will give buyer right to his requirements at

contract price regardless of market price. Citation of Oklahoma case of Sherman M. & L. Works v. Carey, Lombard, Young & Co., 227 Pac. 110. Court held, reversing decision of lower courts to contrary, that, "where the purchase is for a specific purpose and for a particular individual to be used for a certain and definite purpose, all of which facts are made known to the seller . . . wherein purchasers agree to use no other brand of cement. . . . then amount of material necessary to complete job or contract of purchaser, becomes essence of contract, rather than specification wherein certain amount of material is designated, more of less."—JAY BOHN

Trass portland cement. H. RICHARZ. *Zement* (Germany), Feb. 1930, V. 19, No. 6-7, p. 120-3, 144-8.—Contribution to the question of possibilities for use of trass and clinker mixtures. Testpieces were made from portland cement clinker and standard sand (1:3) and from 70 parts of this portland cement, 30 parts of trass and standard sand (1:3, 1.05:3 and 1.1:3). Parallel determinations were made with varying additions of gypsum. Sieve residues of trass cement were higher than those of portland cement. Addition of trass retarded setting process for about two hours. Tensile strength of trass cement was always lower than portland cement except after 90 days water storage. Compressive strength was lower in all cases. Another series of tests was made by grinding raw materials totogether. Tensile and compressive strengths of trass cement were, in nearly every case, higher than strength of portland cement; furthermore trass cement set 1 hour and 20 min. faster than the portland cement. Similar tests were carried out with tufaceous material instead of trass. The strength properties in general were lower and setting slower than in the case of portland cement. Conclusions are drawn for practical use and manufacture of trass cement. Several references are given.—A. E. BEITLICH

Clinker concrete as a lining for the rotary and shaft kiln. O. FREY. *Zement* (Germany), March, 1930, V. 19, No. 11, p. 239.—Clinker of a certain grain size and a suitable softening point can be used with great advantage as lining in cement kilns. The clinker bricks must be stored for at least three to four weeks before being used.—A. E. BEITLICH

Portland cement and pozzuolana. A. POULSEN. *Tonind. Ztg.* (Germany), Feb. 3, 1930. V. 54, No. 10, p. 161-2.—Moler, a Danish diatomaceous earth, on calcining, may be ground with three times its weight of portland cement clinker to form a cement of good strength and of especial usefulness for concrete in contact with sea water and other corrosive waters.—F. O. ANDEREGG

Air separated cement. A. B. HELBIG. *Zement* (Germany), March, 1930, V. 19, No. 11, p. 237-9.—The advantages of the installation of an air separator in the grinding process of a cement mill are demonstrated.—A. E. BEITLICH

MISCELLANEOUS

The effect of sodium and potassium salts on combustion. J. E. DUCHEZ. *Rev. materiaux construction trav. publics* (France), Jan., 1930, No. 244, p. 8-9.—An abnormal fusion of kiln lining was found to be caused by traces of alkalies, indicating that much hotter fire than ordinary had been used. It was found, and confirmed by expts. in several kilns, that addition of 0.1 per cent NaCl to the coal resulted in saving of 15 to 25 per cent of fuel.—F. O. ANDEREGG

Grinding tests with a tube mill without an air separator. GROSSE, FOERDERREUTHER AND RAMMLER. *Zement* (Germany), Feb., 1930, V. 19, No. 9-10, p. 189-94, 214-7.—Experiments were made to find the relation between fineness, power consumption, output, load limit and moisture of the powdered coal. The mill was a three compartment compeb mill ("Solo-mill" of 9 m. length and 1.5 m. diameter. The filling consisted of 2000 kg. steel balls (30-50 mm.) in the first, 2400 kg. steel balls (20-30 mm.) in the second and 700

kg. flint pebbles in the third chamber. Power consumption of empty mill 102 K. W.; speed 27 r. p. m. Coal samples were taken from mill in certain time intervals for sieve analysis. It was possible to calculate the composition of the coal powder from a single test on one standard sieve. Graphs showing the relation between sieve residue and output are given. The influence of the output and power consumption on fineness were studied. Data are furnished about power consumption of elevators, table feeding devices, dust collection and dust transportation.—A. E. BEITLICH

Gypsum and anhydrite as retarding agents. S. STEIN. *Zement* (Germany), March, 1930, V. 19, No. 11, p. 240-1.—Solubility determinations of gypsum and anhydrite in distilled water and lime water were made. Setting time and strength of cements with gypsum and anhydrite were studied, and it was found, that anhydrite has no improving effect on the cement (quick set and low strength). Test pieces were exposed to high pressure at certain time intervals shortly after the mixing and the water pressed from these samples was analyzed. The water from the samples of clinker and clinker with anhydrite showed much higher alkalinity and remarkable lower CaO and SO_3 contents than the water from the clinker with gypsum. The first phase of the retarding process seems to be a neutralization of the alkalinity by the gypsum.—A. E. BEITLICH

PROPERTIES OF CONCRETE

Theory of the expansion of hydraulic cements. HENRI LAFUMA. *Rev. materiaux construction trav. publics.* (France), Dec., 1929, No. 243, p. 441-4; Jan., 1930, No. 244, p. 4-8.—Those reactions that take place during the setting and hardening of cements are probably preceded by solution, while those that result in expansion or disintegration do not need to go through solution stage. Action of $CaSO_4$ on alumina compounds is an example of latter.—F. O. ANDEREGG

Concrete mixing under diminished pressure. ANON. *Tonind. Ztg.* (Germany), Jan. 27, 1930, V. 54, No. 8, p. 131-2—In system developed by Emperger, aggregate is placed in mixer drum which is then filled with water and stirred enough to remove all air. Drum is then closed tightly and excess water is pumped out. Cement in suspension is then introduced and mixing proceeds under reduced pressure. The result is a concrete nearly free of entrapped air. D. R. Pat. has been applied for.—F. O. ANDEREGG

The application of concrete in contact with the ground. WERNER MORITZ. *Tonind. Ztg.* (Germany), Feb. 17, 1930, V. 54, No. 14, p. 223.—The different corrosive substances found in ground waters must be taken into consideration before placing concrete. Fused alumina cement was found generally to be more resistant to corrosive ground waters than portland or slag-portland mixes. Latter might be protected with tile set in tar or asphalt, but danger of the development of shrinkage cracks must not be forgotten.—F. O. ANDEREGG

The testing of plastic mortar. HAEGERMANN. *Tonind. Ztg.* (Germany), March 3, 1930. V. 54, No. 18, p. 296-9.—Proposal is made to substitute for usual tensile test (German) with earth moist mortar, a flexural test with a plastic mortar. Consistency is to be adjusted on flow table, and bars 4 x 4 x 20 cm. are suggested.—F. O. ANDEREGG

The examination of plastic mortars. G. HAEGERMANN. *Zement* (Germany), Feb., 1930, V. 19, No. 8, p. 167-74.—The author outlines a method for the testing of plastic mortars by means of determining the compressive and bending strength. The methods by Ros, Féret and Schuele, which were accepted by the International Congress for Testing Materials, are described

and their disadvantages discussed. Results, obtained by these methods, were varied and depended largely on nature of sand and fineness of cement. A revised method is given to overcome these difficulties by using mixed standard sand (4 parts 1.02-1.50 mm. and 1 part 0.10-0.30 mm.), special determination of the plasticity and exact observation of several manipulations. The test pieces, which are 4 by 4 by 20 cm., are prepared from a mixture of 600 g. cement, 360 g. fine sand and 1440 g. coarse sand. The water for normal consistency is to be determined with a shaking table. The test pieces are broken in a bending apparatus; the broken pieces can be used for determination of compressive strength. Every necessary piece of apparatus is exactly described and its dimensions are given. The results so far obtained with this method are very satisfactory.—A. E. BEITLICH

Hair cracks in concrete. *Das Betonwerk* (Germany), Dec. 15, 1929, V. 17, No. 50, p. 773.—Hair cracks are caused by rich mix, too much tamping, excess water, too much troweling, too quick drying. Heating of concrete mass during setting or hardening is fundamental reason for hair cracks. Heat generated is greater in larger masses and increases with proportion of cement because setting of portland cement is chemical action. As moisture evaporates from concrete masses, shrinkage results. Surface shrinks first forming cracks. Cracks are eliminated by curing masses in warm moist air preferably in an enclosure with wet steam for four or five days. Exact regulation of moisture and warmth is indicated.—H. FRAUENFELDER

Sodium silicate as an addition to cement. (See MATERIALS)

The effect of calcium chloride on concrete. (See MATERIALS)

Trass portland cement. (See MATERIALS)

Internationalization of the portland cement standards—Part II.—(See MATERIALS)

Abutments of a single-span bridge. (See FIELD CONSTRUCTION)

The Arlington curing experiments—a study of the effect of various curing treatments on concrete pavement slabs. (See FIELD CONSTRUCTION)

ENGINEERING DESIGN

BRIDGES

Longest multiple-arch concrete highway bridge to span Susquehanna River. CHARLES M. NELSON. *Eng. & Con.*, Mar., 1930, p. 119-122.—Probably longest multiple-arch concrete highway bridge in the country is now being constructed over the Susquehanna River between Columbia and Wrightsville, Pa. With exception of first span adjoining abutment on each side, spans are on tangent. River structure consists of 28 concrete arches with clear span of 185 ft., carried on 27 piers and shore abutments. Five of river piers are designed to take dead-load thrust during construction. Roadway is 38 ft. wide between curbs. Floor system was designed for load of three 25-ton trucks abreast or in succession and uniform load of 200 lbs. per sq. ft. over remaining loaded area. Arches, girders and piers were designed for uniform load of 200 lbs. per sq. ft. Each arch has three ribs, 17 ft. 9 in. on centers, 7 ft. wide and from 3 ft. 10 in. to 7 ft. deep. Intrados and extrados lines are parabolic. Rise is 27 ft. 4 in. to extrados. All spans have eleven panels. Floor system is carried on spandrel columns. Expansion joints 1½ in. wide placed in first and fourth panels from each pier will completely separate floor system at four points between piers. Railing joints are one

inch wide. Approaches consist of a number of concrete arches.—N. H. ROY

Engineering in five Kansas Counties. WALDO G. BOWMAN. *Eng. News-Record*, Mar. 20, 1930, p. 470-475.—Douglas County designs and builds its bridges. An interesting design used wherever possible for spans up to 75 ft. is that of pony truss. Floorbeams at sub-panel and panel points support continuous concrete floor slab of 7½ in. to 8½ in. thickness. No stringers are required. Overhead clearance for hauling of livestock was factor in substituting pony for through truss, and cost of construction is less.—N. H. ROY

Reinforced concrete railway bridges. R. L. MCILMOYLE. *Con. Const. Eng.* (England), Jan., 1930, V. 25, No. 1. p. 37-45.—Construction of a five-mile widening between Carrickfergus and Whitehead on Larne Line of London Midland and Scottish Railway, Northern Counties Committee, has just been completed, requiring construction of two new overbridges and one single-track underbridge. The underbridge was the smallest of three and carries main line over an accommodation road. Railway crosses road at approximate angle of 68 degrees and clear skew span is 13 ft. Owing to restrictions on headroom depth available from rail to underside of the deck slab was only 4 ft. 3 in., and as standard practice on L. M. S. N. C. C. requires ballasted deck, available construction depth was 18 in. To meet this it was decided to construct abutments monolithic with deck slab and to utilize resistance of former to reduce live load moment at center of latter. Bridge was then designed as two-hinged rigid frame, and steel was provided to take negative bending moments induced at corners. Larger of two overhead bridges carries main road 24 ft. wide over two railway tracks at angle of 24 degrees. Construction was complicated by lack of alternative route. Large span, 69 ft. 6 in. clear on skew, together with small amount of headroom available, eliminated usual beam and girder design. Original scheme contemplated to construct bridge as barrel with curved intrados and variable slab thickness was abandoned because torsion stresses due to skew could not be easily determined. It was found that by varying depths at center and supports a tee-beam could be designed to give required clearances. Structure consists of four two-hinged rigid frames at 9-ft. centers, with deck slab 10 in. thick and curtain walls between legs varying from 14 in. to 8 in. thick. Work was done in two independent halves in order to keep bridge open for traffic. Other overbridge was of the flat-slab type, believed to be first flat-slab bridge in British Isles carrying public road.—D. E. LARSON

Concrete railway bridge at Neuilly-Sur-Marne. *Con. Const. Eng.* (England), Jan., 1930. V. 25, No. 1, p. 85-89.—Owing to increase in traffic in neighborhood of Paris it has become necessary to make Grande Ceinture lines independent of those of Chemin de fer de l'Est by constructing 13 miles of new double-track between Bobigny and Sucy-Bonneul. Most important structure of the new works is the 230-ft. three-hinged arch of reinforced concrete over Marne. Reinforced concrete was selected instead of steel because of resulting economy and possibility of securing more pleasing effect in completed design. Advantage was taken of depth available to make rib very stiff so as to insure all elastic movements taking place at hinges as desired. With ribs of depth employed these members are always subjected to compression. This resulted in high factor of safety, since working stress in the concrete has been taken as 920 lb. per sq. in., while the actual compressive strength of the concrete at 90 days is 3,550 lb. per sq. in. Clear span of arch is 222 ft., distance between hinges is 230 ft., and rise is 24 ft. 3 in. Each half of arch consists of two slabs at intrados and extrados respectively; these slabs are connected by vertical ribs between which there are four stiffeners. Extrados is straight for part of its length; remainder is cubic parabola. Latter portion carries gravel backing, whose weight acts as stabilizing force. Hinges are of concrete

spirally reinforced on Considere principle, and extend across whole width of arch, reducing unit stress on them. Each hinge contains 12 hoops on each side with 14 turns 3 ft. 3 in. in diameter and extending 3 ft. 3 in. along longitudinal axis. Adjacent spirals intersect and widen slightly at the ends. The hinges are reinforced longitudinally by central layer of bars. Concrete in hinges was mixed in proportions of 1,320 lbs. of portland cement to 14 cu. ft. sand and 28 cu. ft. gravel. In remainder of arch, cement was reduced to 770 lbs. with same proportions of aggregates.—D. E. LARSON

Designing the Hyperion viaduct at Los Angeles. MERRILL BUTLER AND A. L. ENGER, *Eng. News-Record*, Mar. 20, 1930, p. 476-480.—Continuous concrete arch structure is composed of two flat 118 ft. arches; two open spandrel arches of 135 ft. span, 22 ft. rise, one 68 ft., eight 48 ft. barrel arches and a series of other arches, all with elastic piers. The 48 ft. arches are three-centered, and are tilted to fit grade, while other arches are four-centered, being compounded at crown and at points halfway between springing and crown. Each group of arches was considered fixed at abutments and at top of pier footing. Mathematical analysis, elastic method, (cf. Hool V. 3) was followed. Analysis of single span gave dead-load, rib shortening, and temperature stresses. Corrections of rib-shortening stresses thus calculated were necessary for flat deep arches. These corrected stresses agreed with results derived from formulas of Howe and others. Beggs' method of analysis by means of models and deformeter gages was used to supplement design studies especially with reference to effects of live load. The mathematical and Beggs' model anaylses were in close agreement for horizontal and vertical thrusts but gave wide variations in moments. Difficulties were encountered in making cardboard models to truly represent the structure. Celluloid is now used. Some unsatisfactory phases of this method are discussed. The conclusion is drawn that the Beggs method permits accurate analysis of structure too complicated for mathematical analysis.—N. H. ROY

BUILDINGS

Design of Lackawanna's reinforced-concrete freight terminal warehouse at Jersey City. M. HIRSCHTHAL, *Eng. News-Record*, Mar. 27, 1930, p. 519-524.—Extensive terminal developments in Jersey City include eight story building 848 ft. by 162 ft. flanked by flat-slab viaducts on both sides. Both viaducts are joined to the building at second story level. Center lines of two of three tracks on south viaduct are directly above center lines of supporting columns. There is no continuity transversely to reduce the moments in the slab. There is a wide overhang on both sides. E-60 loading with 25 per cent impact allowance was deemed sufficient due to expected slow movements on these structures. Slab thickness is 2 ft.; four-way system of reinforcement was employed, drop panels are 10 ft. square and 10 in. thick. Columns are 3 ft. in diameter with vertical bars and spiral hooping. Footing designs are unusual in that concrete piles supported a continuous footing under each longitudinal row of columns. Columns are spaced at 25 ft. centers except at street intersections. Slabs were designed according to method described in the article on Hackensack River bridge, (cf. *Eng. News-Record*, May 17, 1928, p. 779,) except loading is as given above. North viaduct is similar to south but having however, 5 tracks and a platform. Unusually heavy live loads, considered uniform, were used in the warehouse design, being 350 lb. per sq. ft. for the basement, first and second floors, 300 for third and fourth floors, and 250 for fifth to eighth stories. In addition to these loads a 60,000 gal. water tank is supported by four columns. Even though advantage was taken of the provision for reduction of live loads for multiple stories, permitted by the Jersey City code, the maximum column load in basement columns is 2,750,000 lbs. Concrete piles with individual concrete footings, surmounted by continuous reinforced-concrete mat over the whole area, were

substituted for open caisson foundations. Concrete piles were driven for main footings. Pedestals projected above basement floor to provide for very high punching shear. Columns are 4 ft. 4 in. in diameter, spirally and longitudinally reinforced with maximum amount of reinforcement. Composite columns, used on first and second floors, were of 14-in. H-sections inside of 3 ft. 4 in. spirally and longitudinally reinforced concrete column, all set on steel billet to transfer loads to basement columns. Above second floor, columns were spirally and longitudinally reinforced only and varied from 3 ft. 4 in. in diameter to 20 in. diameter. Flat slabs in this building have two-way reinforcement. A notable design feature is that despite many openings not one beam projects below ceiling level except for fire stop.—N. H. ROY

The new custom building at the main railroad station in Stuttgart (Germany). FUCHES-ROELL. *Zement* (Germany), Feb., 1930, V. 19, No. 6, p. 125-6.—Description of a modern concrete building; special consideration is given to the floor constructions.—A. E. BEITLICH

MISCELLANEOUS

Curves for the analysis of rectangular symmetrical concrete beams with axes V-shaped in plan. ST. HESSLER. *Beton u. Eisen* (Germany), Feb. 5, 1930, V. 29, Heft 3, p. 53.—Fixed-end beams with symmetrical loading are considered. Curves give coefficients for bending moments for various proportions of beam section, and for values of horizontal angle from zero to 180 degrees. Two numerical problems illustrate use of curves. Reference is made to article by author (cf. *Beton u. Eisen*, Heft 23, 1927), for treatment of theoretical basis of curves. —A. A. BRIELMAIER

Tests of columns with high percentages of reinforcing steel. R. SALIGER. *Beton u. Eisen*, (Germany), Jan. 5, 1930, V. 29, Heft 1, p. 7-12.—Tests were made at Technische Hochschule in Vienna in January, 1929, on five pairs of columns, reinforced according to Bauer system, in which steel is fabricated into unit frame ready for placing. Columns were 47 in. in length, of which 27.5 in. was of 13.5-in. diameter. The remaining length was taken up by 2 capitals 9.5 in. long and 16.5 in. in diameter. Ends of vertical steel extended 0.4 in. beyond concrete. Longitudinal steel varied from 4 to 9 per cent, spiral 0.5 to 2 per cent. In one pair of columns, perforated metal casing with area 2.1 per cent of core area, was used instead of spirals. Concrete mix was 1:1⅓:2½, (approx.) with water-cement ratio of about 0.8. Average yield point of vertical steel was 96000 lb. per sq. in.; of spiral steel, 74000 lb. per sq. in. Compressive strength of 8-in. concrete cubes tested when 40 days old was 4200 lb. per sq. in. Failure of columns in every case occurred by buckling of longitudinal steel, after concrete fireproofing had broken loose. Lateral reinforcement failed at or after maximum load. Formulas are given for ultimate loads and for allowable loads on columns in terms of concrete, vertical and spiral steel. Distribution of column load between concrete and steel is stated for each test column. Concrete took ¼ to ⅙ of the load and reinforcement took remainder. Safety against failure therefore depends mainly upon steel and is not greatly influenced by strength of concrete. Larger series of tests are planned.—A. A. BRIELMAIER

Reinforced concrete cooling towers. A. T. J. GUERITTE, *Str. Eng.*, (England), March, 1930, V. 8, No. 3, p. 117-125.—Natural draught cooling towers of reinforced concrete are being built in place of timber type because of (1) short life and high upkeep of the timber structures, (2) danger from hidden rottenness in the timber towers, (3) fire danger and necessity for fire protection of timber structures during time of shutdown for cleaning, (4) decreasing efficiency of timber cooling tower with age, and (5) structural limitations of timber towers. Evolution of the concrete tower led first to two designs of polygonal shape, first being of vertical columns and horizontal bracing as framework for wall panels,

and second of precast blocks with mortar joints. Later design of smooth shell of double curvature was evolved and perfected. General shape is hyperbolic in elevation and circular in plan. Considerable detail is devoted to description of design and construction of this latest type of tower, which has permitted a growth in size to height of 215 ft. with base diameter of 175 ft. and a circulating capacity of five million gal. per hr.—V. P. JENSEN

Harbor works at Vlissingen. *Con. Constr. Eng.* (England), Jan., 1930, Vol. 25, No. 1, p. 79-82.—Recommendations of commission appointed to report on the works required to improve the port of Vlissingen included extension of existing harbor by construction of quay-wall 1,150 ft. long, and new lock 115 ft. wide leading to existing basins. Main features of quay-wall are reinforced concrete sheet-piled wall retaining filling, with concrete superstructure founded on reinforced concrete piles driven with a batter. Concrete wall is provided with reinforced concrete platform with tie walls 19 in. thick at intervals of 9 ft. 3 in. Thickness of platform is 27½ in. Additional ties are provided in longitudinal direction. Section of these is 6 ft. 7 in. by 2 ft. 6 in. Space above the platform is back filled with sandy gravel. Extreme width of wall is 75 ft. 6 in.—D. E. LARSON

Reinforced concrete slurry tanks. *Con. Constr. Eng.* (England), Jan., 1930, V. 25, No. 1, p. 69-73.—Group of four slurry-doctoring tanks at Swanscombe Works of Associated Portland Cement Manufacturers, Ltd., provides an example of circular tanks with suspended pyramid bottoms, constructed of reinforced concrete. Upper cylindrical portion of each tank is 26 ft. 9 in. in height and has internal diameter of 17 ft. Diameter decreases to 12 ft. 6 in. at the bottom of wall haunch and is further reduced to 9 in., diameter of outlet. Battery of four tanks is supported on 16 reinforced concrete columns each 2 ft. square. Total height from ground level to top of tanks is 52 ft. Corner columns are carried down to solid chalk on 5-ft. square bases of plain concrete, and interior columns in groups of two, on bases 11 ft. by 5 ft. in plan. Overall length of group of four tanks, arranged in series at 17 ft. 9 in. centers, is 71 ft. 3 in., walls having constant thickness of 6 in. from top to bottom except between each pair of tanks where thickness is 9 in. Walls have been reinforced to resist tension caused by lateral pressure of liquid of 100 lbs. per cu. ft. density. Five figures.—D. E. LARSON

Lining the Beacon Hill sewer tunnel at Seattle. *Eng. News Record,* V. 104, No. 11, p. 442-44.—The 5,580-ft. tunnel now being built under Beacon Hill in Seattle to convey sewage from borders of Lake Washington to Elliott Bay is being lined with concrete circular section with 9-ft. inside diameter. Because of stable quality of clay which tunnel traverses, timber lining, put in as tunnel was driven, consisted of ring made up of 15 segments, and concrete lining is later poured inside this ring. Up to level of spring line, wood panel forms are used and lower half of concrete lining is poured by gravity dump from cars on elevated tracks. For arch, collapsible steel forms are used, and in these concrete is placed by the pneumatic process. After timber lining is in place, a series of concrete blocks is placed at 5-ft. intervals along invert to support wood panel forms used in lower half of tunnel. Concrete is placed in invert by means of small wooden chutes. It is customary to leave forms in place for 48 hours after pouring. For arch, six sets of collapsible steel forms, 20 ft. long were provided. These are left in place for 72 hours after pouring. Concrete mixing plant was erected at east portal and one gasoline and one electric locomotive used for delivery, concrete being taken into tunnel in 1-cu. yd. dump cars. Ordinarily a 1:2¼:3¾ mix is used. Net thickness of concrete lining is 13½ in. Average rate of progress in placing lower half of tunnel lining has been over 60 ft. per day; in arch, about 40 ft. per day.—D. E. LARSON

A method of arch design. G. P. MANNING. *Conc. Constr. Eng.* (England), Jan., 1930, V. 25, No. 1, p. 29-36.—Best shape and thickness of arched member depend on at least ten different

considerations, some of which are not capable of being expressed in mathematical terms. Design, therefore, involves certain amount of trial and error. Some writers assert that best shape for arched member is line of thrust produced by dead load so that arch is in state of pure thrust under dead load alone. Unfortunately it can be demonstrated that it is impossible to bring about these conditions, and it can be shown that, even if it were possible, best shape of arch would not always result. It is clear that designer must have complete range of shapes and thicknesses at his command which he can apply to each particular problem and from which he can, by trial and error, pick out one most suitable. It is often supposed that maximum stresses in middle half of span occur at crown. This is fallacy. In many shapes of arches much larger bending moments may occur at other sections. Analysis must therefore not be confined to one or two sections but must include complete envelope of bending moments giving maximum possible stresses at all points. Computations by usual method are long and tedious. In order to avoid greater part of figuring and to present a correct and easily checked result author has worked out a series of results where shape of arch and its thickness are controlled by mathematical factors. From these designer can get at once a series of "ready made" influence lines or bending moment envelopes making it easy to work out two or three alternate solutions for any particular case.—D. E. LARSON

Operating a public swimming pool. C. E. BARRY. *Can. Eng.*, Feb. 25, 1930, V. 58, No. 8, p. 241.—Swimming pool, 25 x 60 ft., enclosed in frame structure was built upon continuous concrete footings with side walls of concrete to height of 18 in. above floor. Floor and side walls of pool were of 1-2-4 concrete heavily reinforced, without the use of any waterproofing. Floor thickness was 10 in. and side walls 12 in. Article includes detailed descriptions of lighting, heating, filtration and mechanical equipment as well as results of operating tests.—G. M. WILLIAMS

Municipal swimming pool at Hamilton, Ont. E. H. DARLING. *Can. Eng.* March 11, 1930, v. 58, No. 10, p. 311.—Concrete swimming pool with tank 75 ft. x 45 ft. surrounded by grandstand for accommodation of 1200 spectators, with locker rooms, lavatories, showers and mechanical equipment below has been built at Hamilton in preparation for the British Empire games. Walls of pool are 10 in. thick, reinforced, concrete proportions about 1:1½:2, the coarse aggregate crushed limestone 1 in. to ⅜ in. Three pounds of "Aquagel" waterproofing per bag of cement was used. After testing tank for watertightness, white enameled tile lining was supplied with longitudinal rows of black tile to mark lanes for racing. Underwater illumination consisting of 20 300-watt lights placed below water level behind thick glass portholes adds to safety in operation. Structure surrounding pool consists of exterior walls of 4 in. face brick backed by cinder blocks, and steel truss roof. All interior partitions are of cinder tile. —G. M. WILLIAMS

Construction of storage silos in modern crushing plant. DR. E. LEE HEDENREICH, *Rock Products*, Feb. 15, 1930, V. 33, No. 4, p. 72-74.—Present applications of lateral and vertical pressures in reinforced concrete bins for storage silos in stone crushing plants take into consideration pressures of crushed stone of different size and weight in computations of circular or square concrete storage bins for such materials. Crushed rock comes under "Fragmentary Materials" of C. C. Williams, A. S. C. E., classification of non-solids and non-liquid materials, "—in which, after certain depth, there is no lateral pressure, for material acts as solid after rough particles are held together by intercogging under load." Determination of lateral pressure has hitherto been considered as dependent upon the angle of repose, ϕ, and the angle of friction against the bin wall, ϕ'. Assumption that these two angles are same in case of different sizes of crushed stone is erroneous. Ratio k between lateral and vertical pressure at wall is more or less a function of ϕ and tangent of angle ϕ'.

Formula for computation of lateral pressure is presented and method of calculating vertical pressures on top of tunnel at bottom of bins illustrated. Four tables include design data and two diagrams illustrate application of formula.—JAY BOHN

Central electricity board's report. *Engineer* (England), Apr. 11, 1930, V. 149, No. 3874, p. 400.—In annual report of the C. E. B. it was stated that during 1929, testing of towers was carried out and designed to reproduce all conditions likely to be encountered in practice. Towers were tested to destruction, and tests on lighter towers were taken to indicate reliability of method of design for larger structures, such as at river crossings. Electrical and mechanical tests were also made. On Brighton to Worthing line, spun reinforced concrete supports are employed. A crossing of the Thames, which will necessitate a main span of 3,000 ft. and two 500 ft. high towers, is contemplated.—JOHN E. ADAMS

Swimming bath at Stoke Newington, London N. *Arch. Jour.* (England), Apr. 9, 1930, V. 71, No. 1838, p. 569.—New swimming bath, in reinforced concrete, contains 127,000 gallons of water, which is circulated through purifiers every 4 hours. Water is treated with sulphate of alumina and soda ash and is delivered into air-cleansed pressure filters. In leaving filters water is reheated to 74° F. in tubular calorifiers and re-oxygenized by contact with compressed air in an enclosed type aerator. Bacteriological purification is ensured by a minute trace of chlorine administered by chloronome. Bath hall is of reinforced concrete, rubbed and colored with distemper. The pool is faced with blue and white glazed blocks, and walls below gallery with terrazzo. Promenade around bath is of vitrified glass cubes. Semicircular arch ribs carry the roof and suspended ceiling, and spacious corridors and rooms are provided.—JOHN E. ADAMS

Pole Footings. JAMES CYRIL STOBIE. *Jour. Inst. of Eng.* (Australia), Feb. 1930, V. 2, No. 2, p. 58-63.—Tests have been made by The Adelaide Electric Supply Co. to ascertain ground resistance to overturning of electric power and telegraph line supports with various types of pole foundations. In closely populated areas, where guying is impossible, the overturning moment on the pole is resisted by the horizontal passive earth pressure on two bearing blocks, secured to pole and which distribute force of the overturning couple over a sufficiently large area of ground. Concrete foundations are used where ground is poor or side loadings heavy. Blocks are designed as double cantilevers, taking distributed loading, and tension rods are turned to resist shear. In swampy ground 3 ft. dia. concrete pipes are used as caissons up to 12 ft. length for quick sand. As in some cases sand cannot be reduced below 4 ft. from bottom of pipe, water is allowed to rise and 12 in. layer of concrete run down a tube to cover sand and seal bottom. Pole is erected in pipe on following day, and concreted in after pumping out water. For strength alone there is no great advantage in setting timber poles in concrete, as ground is usually strong enough to hold pole against safe loading for material of pole. Various types of designs of footings and also working foundation loads are tabulated. For consolidated earth, allowable pressure found by test to give no deflection at 18 in. is 3,000 lb. per sq. ft. and 5,400 lb. per sq. ft. at 5 ft. depth, both figures being much in excess of theoretical results. Intensity of pressure at bottom of pole was found much less than would be expected from moments of applied forces. Friction has been thought responsible.—JOHN E. ADAMS

Experiments with reinforced concrete twin-sleepers. R. OTZEN. *Zement* (Germany), Feb., 1930, V. 19, No. 8-9, p. 174-7, 196-9.—Different kinds of sleepers of reinforced concrete were tested in the laboratory for their behavior when exposed to different continuous and intermittent pressures. The impressions, elastic and permanent changes in form, were measured. The twin-sleeper showed 50 per cent less changes in form than a transverse sleeper. The pressure and the deformation of the road bed is very much

decreased. Data are given for calculation of "form-factor" and best sleeper dimensions.—A. E. BEITLICH

The German reinforced concrete standard specifications of 1925. "Die Deutschen Eisenbetonbestimmungen von 1925." ERNST RAUSCH. 1929, Julius Springer, Berlin, R. M. 4.80. Reviewed in *Zement* (Germany), Feb., 1930, V. 19, No. 7, p. 157.—Book describes the different methods for calculation of reinforced concrete structures. All kinds of slab constructions are discussed and data are given about the factor of safety in concrete construction work.—A. E. BEITLICH

Pumping station in Wessermuende (Germany). MANGEL. *Zement* (Germany), Feb., 1930, V. 19, No. 9, p. 200-1.—Description of a modern pumping station in reinforced concrete. The foundations rest on 11m. long concrete piles. The structure under earth is protected by bricklining and asphalt coatings.—A. E. BEITLICH

ARCHITECTURAL DESIGN

The correct surface treatment of mass concrete. *Das Betonwerk*, (Germany), March 2, 1930, V. 18, No. 9.—Concrete as it comes from forms is generally considered uninteresting. Cutting surface with tools, and stuccoing have not given entire satisfaction. It is mentioned that the newly discovered American method known as Contex method exposes beauty of aggregates. Several interesting illustrations are shown.—H. FRAUENFELDER

FIELD CONSTRUCTION

BUILDINGS

Structural haydite concrete in flat slab industrial building. *Concrete*, March, 1930, V. 36, No. 3, p. 13. JOHN BAPTISTE FISCHER.—Haydite was used as coarse aggregate for structural concrete in moderately heavy industrial building intended for extension to the Chicago Screw Company's manufacturing plant at Chicago. Building is of flat-slab construction, reinforced in two directions. Addition constructed during 1929 consisted of four-story upward extension of existing one-story building having high basement. Original structure had been designed for live-load of 250 lb. per sq. ft., with the further expectation that the future concrete would be ordinary stone or gravel concrete weighing about 150 lbs. per cu. ft. Reducing design live load of addition from 250 to 150 lbs. per sq. ft. and using haydite as coarse aggregate, permitted addition of four stories instead of two. Proportions were of 1 sack of cement, 2.1 cu. ft. of sand and 3 cu. ft. of haydite, with 6.4 gallons of water to sack of cement. Two lbs. of "Colloy" admixture per sack cement were used.—C. BACHMANN

How to save in concrete form work. A. B. MACMILLAN, *Concrete*, March, 1930, V. 36, No. 3, p. 17.—Form panels on the sides of interior wall columns are not all of same length. Panel on the inside face of wall column is usually of same length as form for an interior column in the same story. Wall columns are generally rectangular, so consequently on many buildings form panels intended for inside and outside faces of column may be used number of times, without being changed, except where story heights vary.—C. BACHMANN

Economic construction methods. "Vom wirtschaftlichen Bauen." RUDOLF STEGEMANN. Germany, Oskar Laube. V. 5, R.M. 4.00 and V. 6 R.M. 6.00 Reviewed in *Zement* (Germany),

Feb., 1930, V. 19, No. 7, p. 157.—Vol. 5 discusses the different solid ceiling constructions, their properties in regard to fire resistance, sound and heat economy. Vol. 6 deals with questions of building construction, special consideration is given to the use of different building materials from the economic view point.—A. E. BEITLICH

MISCELLANEOUS

Experiences with the ramming of reinforced concrete piles. O. S. *Zement* (Germany), Feb. 1930, V. 19, No. 9, p. 201-3.—The method of testing the supporting power of an unknown ground by ramming only one test pile cannot be recommended. The stresses in the ground, which arise when the piles are driven close together, must be taken into consideration. The following points must be observed; the ground must be tested much deeper than the necessary foundation depth in order to determine smallest possible distance between piles without overloading. It is advised to ram the piles again after several weeks when all stresses are neutralized. In all cases it is of advantage to test ground with two piles spaced not far apart.—A. E. BEITLICH

The foundations of a factory and warehouse in Barmen (Germany). *Zement* (Germany), Feb., 1930, V. 19, No. 7, p. 151-7.—The ground for the heavy structures was very unfavorable, including limestone and dolomitic rocks with layers of soft clay. All kinds of different foundations in reinforced concrete were necessary. A detailed description of the construction and the static calculations are given.—A. E. BEITLICH

Concrete constructions with sliding forms. LANGENBECK. *Zement* (Germany), Feb., 1930, V. 19, No. 6, p. 127-9.—A description of concreting work with sliding forms and its application for the construction of large storage bins and houses.—A. E. BEITLICH

Device for the automatic temperature regulation in the water tanks for test pieces. E. PROBST AND KARL E. DORSCH. *Zement* (Germany), Feb., 1930, V. 19, No. 7, p. 143-4.—A heating apparatus, operated by thermoregulator and a relay is described. Electric bulbs serve as heating units and a stirrer keeps the water in motion. The thermoregulator is filled with benzene which expands and contracts very much with changing temperatures.—A. E. BEITLICH

Abutments of a single - span bridge. ANONYMOUS *Zement* (Germany), Jan., 1930, V. 19, No. 3, p. 58-60.—Description of construction of abutments for a bridge of a coal mine under especially unfavorable surface conditions. Ground was expected to sink slowly about 8 ft. on account of the undermining. It was necessary to place the foundations 25 ft. below the surface since upper layer consisted mostly of soft sandy clay. It was necessary to excavate 4460 cu. meters ground in order to make two excavations 16.3 by 14.8 and 13.0 by 13.5 m. Heavy copper-steel sheet piles of 5 and 6.5 m. length were driven through soft material. Horizontal frames of heavy timber were built in for distance of 1.20 m. Excavated material was transported over a wooden bridge 120 m. long and served as fill for a dam structure. Concrete was transported over same bridge. The abutments rest on reinforced concrete foundations 1.50 and 2.00 m. thick. Brick lining and asphalt isolation protect structure against aggressive waters. Heavy reinforced concrete beams, 1.10 m. long, distribute the stresses to the foundations. Ordinary portland cement (1:8) was used for the main structure of the abutments. The outer sides of the abutments were provided with a 60 cm. layer of broken rocks and a groove, connected to a concrete drain pipe 15 cm. in diameter, serves for draining off water.—A. E. BEITLICH

New methods of making concrete. VON H. MIKELER. *Das Betonwerk* (Germany), March 2, 1930, V. 18, No. 9.—It is noted that in the last five years grading of aggregates, water cement ratio and proper handling of the entire mixture are important, and the worker should put this knowledge

into practice. The main results obtained by use of this knowledge are greater use of cement and better quality concrete. Grading and accurate measurement of sand, aggregates, cement and water by use of instruments and machinery are recommended. Various machines are described and illustrated. Formulae for grading of sand and aggregate are described. Profusely illustrated.—H. FRAUENFELDER

"Tru-Mix" concrete is prepared in three Seattle plants of pioneer Sand and Gravel Co. FRANCIS CHURCH LINCOLN. *Pit and Quarry*, March 26, 1930, V. 19, No. 13, p. 48-50.—Sand and gravel are received at three plants by barges from company's five pits. At plant 1, barges are unloaded to stockpiles or bins. Sand and gravel are weighed for each batch upon springless dial scale, and cement is measured by sackful. Water is measured from tank with gauge-glass on outside. Batch is mixed in 2-cu-yd. mixer, for one minute (unless otherwise specified). Batch counter tallies the concrete as discharged to trucks. No. 2 plant contains 1-cu. yd. and 2-cu. yd. mixers General practice is same as in plant No. 1 except that concrete, after mixing is hoisted in skip before it can be dumped to trucks. At plant 3, mixer is a 1-cu. yd. unit, discharging directly into trucks by gravity. Eight illustrations.—A. J. HOSKIN

Ready-mixed concrete. STANTON WALKER, *Rock Products*, Feb. 1, 1930, V. 33, No. 3, p. 87-90.—Complete presentation of present status of preparation and sale of ready-mixed concrete summarized as follows: (1) Ready-mixed concrete industry gives promise of becoming stabilized business and therefore does not offer prospects of more than conservative return on investment. (2) Field is sufficiently broad to include all types of concrete construction and development opportunities unlimited. (3) Only careful study of building program of locality will determine potential support of plant, 50,000 population usually being minimum requirement. (4) Availability of raw materials is of first importance. (5) Measured by ordinary business practice, gross sales must be relatively large compared with investment. (6) Determination and selection of equipment depends upon local requirements of plant. (7) Reasonable distances of transportation with present equipment have no deleterious effect on quality. (8) Ready-mixed concrete operator is ideally situated to exercise equivalent of laboratory control over concrete used in ordinary construction. Bibliography included.—JAY BOHN

Airport construction and maintenance. W. A. HARDENBERG, *Rock Products*, Feb. 1, 1930, V. 33, No. 3, p. 62.—Airport construction and maintenance in 1930 will afford greater field than ever before for construction industry. The work necessary in the construction of a modern airport includes paving and construction of runways, drainage, water supply, sewerage, lighting, power and housing. In addition, in many instances airport itself will be nucleus about which other construction will center. Increased air travel and heavier planes demand larger and better facilities. Increase of 51 per cent in number of projected airports is noted. Average of 17 cities shows cost of construction to be $2.50 per capita. Paved runways are in demand. Water supply, sewerage, lighting, and housing, because of usual distance from municipal connections, are all costly, but of prime importance. Nearly 1400 airports to be constructed in 1930.—JAY BOHN

Construction methods used in enlarging Cheyenne's filter plant. *Concrete*, March 1930, V. 36, No. 3, p. 22.—When work began on Cheyenne's new filter plant, one wall of old reservoir was uncovered and given 1-in. coat of gunite, having been found sound throughout. New walls were built in three lifts, using wood forms of short sections. Walls are uniformly 7 in. thick and reinforced with ½-in. bars placed on 12 in. centers in two directions. Since slope of wall is ½ to 1, only difficulty was to keep reinforcing correctly in place during placing operations. Lower lift of wall was designed with notch for the floor. Reinforcing steel from wall, in form of bent bars,

was carried out of notch, to be incorporated in the floor slab. Subgrade was carefully prepared and system of supporting beams was laid off, gridiron fashion. Trenches for the beams were carefully cut into the subgrade, no form work being used for floor system. Concrete for both floor slab and beams was placed simultaneously. Only form work required was in headers around section being placed. Each side of panel being placed was made a construction joint and deformed tongue and groove plate used for form, reinforcing bars being carried through plate, as in road work. Aggregates were obtained from nearby pit and were separated into gravel and sand by screening. Both materials were washed and no coarse aggregate larger than 1¼ in. was permitted. Plain concrete in foundations and footings was 1:2½:5 mix. By keeping water content low this mix gave good results and effected saving in materials. The 1:2:4 concrete was used for all reinforced work except in some isolated cases where 1:2½:5 mix was used.—C. BACHMANN

ROADS AND PAVEMENTS

The Arlington curing experiments—a study of the effect of various curing treatments on concrete pavement slabs. L. W. TELLER AND H. L. BOSLEY, *Public Roads*, Feb. 1930.—Tests were made at Arlington Experimental Farm, Virginia, to develop information regarding merits of various methods of curing concrete pavements. Concrete slabs, each 200 ft. long by 24 in. wide by 6 in. in depth, were constructed, of typical paving mixture of same composition throughout. Variables were introduced to supply information on effect on concrete pavements of such factors as: Moisture in subgrade; covering subgrade with tar paper; surface curing with earth, straw, burlap, bituminous materials, calcium chloride and sodium silicate; curing with calcium chloride mixed integrally; slab length; various amounts of steel reinforcement. Effects of these factors on different sections were observed over period of more than two and one-half years, and most important information was obtained from the following observations and tests: Transverse cracking of test sections; local shrinkage cracking; visual inspection of slab surfaces; tests for surface hardness; loss of moisture from concrete during curing period; tests on control specimens; subgrade friction; effect of steel reinforcement on volume change. Resulting indications are: (1) Careful attention to curing improves quality of concrete both from standpoint of strength and of surface appearance. (2) Any curing method, to be effective, must commence as soon as concrete surface is finished. (3) Wet-burlap-wet-earth curing method was more effective than any of other methods used. (4) Immediate application of wet burlap largely prevented surface checking so prevalent in slabs where curing was delayed. (5) Under adverse conditions concrete may lose 40 to 45 per cent of original mixing water during first few weeks after placing and, more important, under these conditions 75 per cent of this loss may occur during first 24 hours. (6) Application of black surface to concrete slabs causes greater temperature variations and consequent volume changes than would otherwise result and may result in abnormal amount of transverse cracking. (7) Use of transverse joints to allow free longitudinal movement of slab ends is of material benefit for control of transverse cracking. (8) Method used for curing concrete pavement may affect its surface hardness. (9) Increasing percentage of continuous longitudinal reinforcing steel decreases the average slab length.—F. H. JACKSON

An exposed aggregate drive. ALB. L. BITTER. *Concrete*, March, 1930, V. 36, No. 3, p. 35.—Design for residential driveway provided for one entrance branching into a 9 ft. drive to garage and loop to parking space. Except at the flared portion, entrance part of drive is 17 ft. wide. Concrete is 6 in. thick, bottom course being 4 in., proportioned 1:2:3, with 2-in. top of 1:1:3 mix. Color is in top course only, latter containing an excess of coarse aggregate. Wire mesh reinforcement weighing 42 lbs. per hundred sq. ft. was placed between

top and bottom courses. Expansion joints ½ in. wide were installed at 33 ft. intervals, and slabs joined by steel dowel bars spaced 3 ft. on centers. Red color was used, at rate of 6 lbs. per sack of cement. Concrete was mixed in batch mixer. Top course was struck off and floated with wood float. Mississippi River gravel ranging in size from ½ to ¾ in. was put in mixer, without mortar, and drum turned to coat pebbles with color. Pebbles were then cast on the slab and tamped into surface. From 2½ to 5 hrs. after concrete was laid, surface aggregate was exposed by brushing out mortar with ordinary house brooms, and flushing with water to carry away loosened material and clean surface. Commercial curing paper was used.—C. BACHMANN

The new custom building at the main railroad station in Stuttgart (Germany). (See ENGINEERING DESIGN)

Lining the Beacon Hill sewer tunnel at Seattle. (See ENGINEERING DESIGN)